

WISCONSIN HIGHWAY RESEARCH PROGRAM

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**REHABILITATION TECHNIQUES FOR
CONCRETE BRIDGES**

DRAFT FINAL REPORT

by

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16. Abstract This research project addresses rehabilitation techniques for reinforced and prestressed concrete bridges, focusing primarily on corrosion of prestressed concrete beam-ends. An extensive literature database on repair of concrete bridges was developed using Microsoft Access. A limited, first version of an expert system computer program, Concrete Bridge Assessment and Rehabilitation (ConBAR), was developed to assist in diagnosis of concrete bridge deterioration problems and to identify repair, rehabilitation, or preventative maintenance options. The effectiveness of several repair methods in mitigating corrosion damage and providing protection to prestressed concrete beam-ends was evaluated experimentally. These methods included silane sealers, epoxy coatings, patching, polymer (resin) coatings and fiber-reinforced polymer (FRP) wraps.					
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EXECUTIVE SUMMARY

This research project addressed rehabilitation techniques for reinforced and prestressed concrete bridges, focusing primarily on corrosion of prestressed concrete beam-ends. The primary objectives of this research were: (1) to collect and synthesize information on rehabilitation methods for concrete bridges (2) to evaluate the effectiveness of preventative and corrective methods to address deterioration of prestressed concrete beam-ends and (3) to initiate development of an expert system software program to assist in the assessment, diagnosis, and repair of concrete bridges.

A comprehensive review of available literature in the field of rehabilitation of concrete bridges, especially in northern climates, was performed. The results of this review are summarized in this report. In addition, an extensive literature database on repair of concrete bridges was developed using Microsoft[®] Access. Information on a total of 570 papers and reports are included in this searchable database.

An initial version of an expert system computer program, Concrete Bridge Assessment and Rehabilitation (ConBAR), was developed to assist in the diagnosis of concrete bridge deterioration problems and to identify repair, rehabilitation, or preventative maintenance options. This program includes a user-friendly interface that obtains relevant information on the subject bridge through a series of questions, and provides suggestions and recommendations to the user. The depth and variety of questions that ConBAR asks the user before making recommendations far exceed the scope of previous attempts at developing such expert system tools for concrete bridges. This necessitates a very large set of expert rules

(based on combinations of possible answers) that must be incorporated into the program. This program currently includes the complete infrastructure required as well as a limited number of expert rules, which must be expanded and enhanced in future developments of this program.

Based on the results of the literature review, a test plan was developed to address corrosion-induced damage and subsequent repair of beams-ends due to chloride-laden water infiltrating through faulty expansion joints. This problem was selected for experimental evaluation because of its prevalence in northern states such as Wisconsin, and the lack of proven methods to address them. The effectiveness of several preventive solutions/repair methods in mitigating damage and providing corrosion protection was evaluated experimentally. These included localized applications of silane sealers, epoxy coatings, patching, polymer resin coating, and fiber-reinforced polymer (FRP) wraps.

A total of five 36-inch-deep, 8-ft-long prestressed concrete beam specimens were fabricated and tested. The two ends of each beam were either left untreated or were treated using different protective materials and procedures. The beam-ends were subjected to wet/dry cycles of salt-water sprays together with imposition of an impressed electric current to accelerate the corrosion process. After an initial exposure period of 6 months, some of the previously untreated beam-ends were also repaired/protected. The accelerated corrosion process was then continued. The total exposure period for all specimens was 1-½ years. A series of tests were performed during the exposure period. These included half-cell potential measurements, corrosion current measurements, strain measurements, and chloride content measurements. At the conclusion of testing, the end regions of the test specimens were partially dissected to visually examine the state of corrosion of strands.

At the conclusion of the experimental program, an evaluation of various treatments was made. These evaluations were based on the extent of cracking observed, measured chloride penetrations, and observed extent of corrosion during dissection. The best solution is determined to be treating the beam-ends from the first day, i.e. before installation in the field. The treatment area would be limited to all surfaces within a 2-ft-length at the two ends of each beam. This includes the back end surface and the bottom surface. When the strands are cut flush with the back of the beam, the treatment must cover the cut end well to prevent horizontal migration of chlorides through interstitial spaces between wires. In cases where the strands are not cut flush (i.e. embedded in the diaphragm concrete), the exposed strand must be coated well to prevent horizontal chloride migration.

This approach (treatment from the first day) is far more effective, and easier, than subsequent treatments in the field. The carbon fiber-reinforced polymer (FRP) coating, and polymer resin coating (FRP without fiber) were found to be the most effective treatments. Epoxy coating was the next best solution followed by silane treatment. As expected, leaving the beam-end untreated resulted in the worst overall performance.

Considering that the FRP wrap, polymer resin coating, and epoxy coating were generally effective, it is recommended that either polymer (resin) coating or epoxy coating be used in new construction to protect the prestressed concrete beam-ends. The FRP wraps did not significantly improve performance over polymer resin coating, and would only add to the cost and difficulty of treatment. Since protecting the end face of the beam and the cut ends of the strands are crucial, it is recommended that such treatments be performed in advance of installation in the field. The presence of diaphragms, bearings or other obstructions would

likely make the field application of coatings to the beam-ends very difficult; especially after the diaphragm and deck concrete is cast.

For existing prestressed concrete beam-ends, it is recommended that the protective treatments be applied as soon as possible, before chloride levels increase significantly. It is expected that the applications of polymer resin coating or epoxy-coatings to the exposed surfaces of the beam-ends in the field would contribute, albeit not as effectively, to the protection of beam-ends in the long run, if such treatments are implemented before chloride contaminations and corrosion have taken hold. In such cases, all exposed surfaces should be treated with either polymer resin coating or epoxy coating. The extent of pre-existing chloride contamination can be measured in the field (on the bottom flange at about 2 inches from the end of the beam) and compared against chloride contents measured in areas not exposed to chloride contaminations.

In cases where corrosion and damage is advanced and has resulted in cracking and spalling of the beam-ends, the conventional patching alone would likely not be a durable repair method. Although not tested in this experimental effort, a patch repair that is subsequently coated with polymer resin coating or epoxy coating would likely provide a more effective repair.

Although the above results and recommendations were based on tests on beam-ends, it is expected that they would also be applicable to pier elements (such as pier caps and columns) and abutments.

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INTRODUCTION

1.1 PROBLEM STATEMENT

A number of concrete bridges in Wisconsin and elsewhere have shown signs of deterioration due to aging, corrosion, and other detrimental factors. Although bridges are generally expected to yield a service life of 50 to 100 years, some bridges are exhibiting signs of distress at a much younger age. Increased traffic requirements, the use of deicing salts, and lack of adequate preventive maintenance programs contribute to deterioration of existing bridges. Considering the enormous cost and effort required to remedy bridge deficiencies, it is crucial that a concerted effort be made to identify practical, effective and economical methods for repair and rehabilitation of bridges. This research project addresses repair and rehabilitation techniques for reinforced and prestressed concrete bridges, focusing primarily on corrosion of prestressed concrete beam-ends.

1.2 BACKGROUND AND SIGNIFICANCE OF WORK

The most prevalent cause of deterioration in concrete bridges is corrosion. Diffusion of chloride ions through concrete can destroy the passivity of steel and initiate the corrosion process. In northern climates, such as Wisconsin, the primary source of chlorides is found in deicing salts used to melt snow in the winter. Bridges in northern climates are susceptible to corrosion in a manner different from bridges located in warm coastal climates. In such climates, girders are prone to corrosion mainly at the end regions (Figures 1 and 2). This is the

result of water leaking through faulty expansion joints and then reaching the girder ends. Improper drainage can also allow salt water to penetrate into other parts of the superstructure, including fascia girders. The resulting steel corrosion and the spalling of concrete, can cause irreversible damage to beam-ends. Rehabilitation of damaged beam-ends generally requires the complete removal of the damaged region, followed by reconstruction. Issues with this repair procedure include reoccurring spalls due to inadequate bond between the new and existing concrete. In addition, these types of repairs may not be very effective in the long term as contaminants in areas adjacent to the repair can, overtime, migrate to the repair region. This effect is more pronounced if drainage issues are not corrected. Since this type of damage is frequently encountered in northern states such as Wisconsin, the effectiveness of various traditional and state-of-the-art repair techniques are investigated.



Figure 1. Damaged Beam-Ends



Figure 2. Close-up of Beam End

1.3 OBJECTIVES

The primary objectives of this research were: (1) to collect and synthesize information on repair and rehabilitation methods for concrete bridges (2) to evaluate the effectiveness of preventive and corrective methods to address deterioration of prestressed bridge beam-ends

and (3) to initiate development of an expert system software program to assist in the assessment, diagnosis, and repair of concrete bridges.

1.4 SCOPE OF WORK AND STUDY APPROACH

The scope of this research included: (1) a thorough literature review of concrete bridge rehabilitation techniques, (2) evaluation and testing of a number of preventive and repair regimes, (3) development of a basic form of an expert system software program and (4) preparation of a final report.

A thorough understanding of the state-of-the-art in the field of rehabilitation of concrete bridges, especially in northern climates, was considered crucial for the success of this effort. Therefore, a comprehensive review of available literature in relevant subject areas was performed. On-line sources of information, as well as conventional search databases were utilized.

An extensive literature database was developed using Microsoft® Access. Over 570 papers were catalogued. They include such searchable information as the title, publisher, author, and date. The database also includes the abstracts or summaries of many of the papers. The user can search the database by performing a keyword, title, or author query.

Based on the results of the literature review, a test plan and repair concept were submitted and approved by the Project Oversight Committee, appointed by the project's sponsor, the Wisconsin Department of Transportation (WisDOT). The work plan included performing laboratory tests on five new 8-foot long prestressed concrete bridge I-beams to address

corrosion-damage and subsequent repair of beams ends due to chloride-laden water infiltrating through faulty expansion joints.

The beam-ends were subjected to wet/dry cycles of salt laden water to accelerate the corrosion process. In addition to the salt-water exposure, the beam-ends were subjected to an impressed electric current to assist in accelerated corrosion. Two “cathode” bars were placed in the beam and the entire reinforcement system (strands and bars) was made anodic. This creates a “reverse cathodic protection” system, thus accelerating corrosion. Some end regions were pretreated with a sealer, epoxy coating, polymer (resin), or fiber-reinforced polymer (FRP) composite wrap to assess their effectiveness in protecting the beam when subjected to an accelerated corrosive environment.

Several repair schemes were also implemented to evaluate their effectiveness in reducing corrosion and preventing further damage. As was done initially, sealer, epoxy coating, polymer (resin), and FRP wrap treatments were also applied after an initial exposure period of 6 months. In addition, one beam-end was patch repaired with no additional protection system to compare its performance with other systems. After the repairs were completed and the surface treatments applied, the beam-ends were again subjected to an accelerated corrosion regime. Finally, the protection systems were evaluated to determine which system(s) provided corrosion mitigation and the best corrosion protection.

An initial version of an expert system computer program, Concrete Bridge Assessment and Rehabilitation (ConBAR), was developed to assist in the diagnosis of concrete bridge deterioration problems and to identify repair, rehabilitation, or preventative maintenance options. This program includes a user-friendly interface that obtains relevant information on

the subject bridge through a series of questions, and provides suggestions and recommendations to the user. The depth and variety of questions that ConBAR asks the user before making recommendations far exceed the scope of previous attempts at developing such expert system tools for concrete bridges. This necessitates a very large set of expert rules (based on combinations of possible answers) that must be incorporated into the program. This program currently includes the complete infrastructure required as well as a limited number of expert rules, which must be expanded and enhanced in future developments of this program. It is important to emphasize that the tools developed are intended and expected to assist and facilitate the work of experienced maintenance personnel, and not to replace it.

LITERATURE REVIEW

2.1 INTRODUCTION

Deterioration of bridge superstructure and substructure elements is a common problem in the United States. A large number of bridges in the United States were built after the Second World War. Some of these bridges were not designed to withstand the current environmental and traffic requirements, and consequently are experiencing significant distress. Deterioration in bridges can take several forms and stem from various causes. Among the causes are corrosion, structural damage from vehicle impact, and deficiencies in the original design and construction. Methods of repair are numerous and they range from simple spot patching to more complex repair regimes. Since a significant number of bridges are considered deficient or obsolete, economical ways must be found to improve the infrastructure condition. Although complete rebuilding is sometimes deemed necessary, repair and rehabilitation can be far more economical when the methods are effective.

Corrosion of concrete bridge elements is a significant and costly concern due to the possibility of premature deterioration. “The annual direct cost of corrosion of highway bridges is estimated to be between \$6.43 billion and \$10.15 billion. Life-cycle analysis estimates the indirect costs to the user, due to traffic delays and lost productivity, at more than 10 times the direct cost of corrosion” [54]. Therefore, current rehabilitation methods must be evaluated to determine their performance and cost effectiveness. A number of studies have concluded that traditional repair schemes (i.e. concrete patching) lack longevity and are susceptible to

continued deterioration. State-of-the-art materials and procedures (i.e. fiber reinforced composites) have shown in some studies to be an effective alternative for repairing corrosion-damaged concrete. This literature review briefly summarizes traditional and state-of-the-art procedures used to repair corrosion damaged bridge elements.

2.2 CORROSION MECHANISMS

Concrete is normally durable in moist, oxygen rich environments, but steel can be unstable under these conditions. Concrete provides a protective environment to embedded steel by supplying a physical barrier to the ingress of deleterious substances as well as a chemical protective shield. If the physical integrity of the concrete is altered, the protective capability of the concrete barrier is reduced. Protection is also provided to the reinforcing steel by the high alkalinity of the surrounding concrete. The high pH (12 to 13) of the pore water in concrete provides a natural passive chemical environment for reinforcing steel. As concrete ages, environmental exposure can lead to the breakdown of the passive layer. Corrosion would occur if the passive layer is destroyed and sufficient amounts of oxygen and moisture are present. Presence of chloride ions or carbonation can damage the passive layer and accelerate the corrosion process significantly.

The corrosion process is electrochemical in nature. It is driven by the appearance of cathodic and anodic regions on the metal surface (see Figure 3). This can be attributed to different chemical concentrations or the varying availability of oxygen or moisture at different locations along reinforcing bars. At the anode site iron is dissociated to form ferrous ions and electrons. The electrons migrate toward the cathodic site where the ferrous ions dissolve in the concrete pore solution. At the cathodic site, oxygen in the pore solution combines with the electrons to

form hydroxyl ions. The ferrous and hydroxyl ions move in opposite directions through the pore solution, when they combine, ferrous hydroxide is precipitated. The precipitated corrosion products occupy a larger volume than the non-corroded steel. As the concentration of corrosion products increase, an increasing pressure is exerted on the concrete until it cracks and eventually spalls [26].

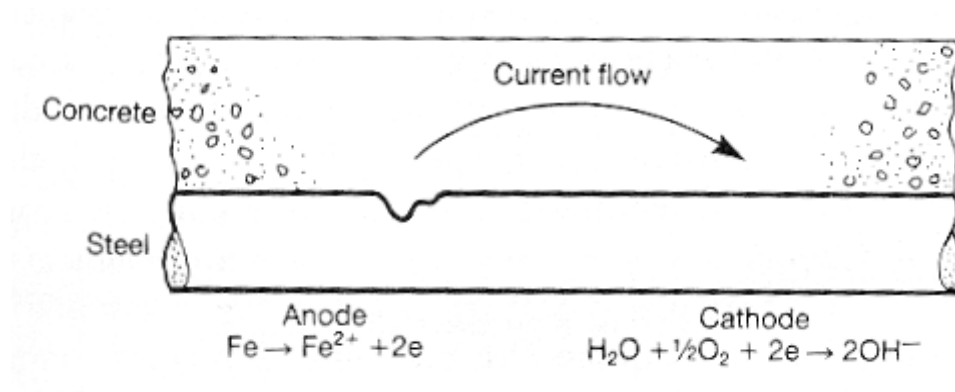


Figure 3. Schematic of Electrochemical Corrosion Process [31]

Pitting corrosion may result if the chloride concentration is highly localized. During pitting corrosion small pits, or holes, form on the steel surface. The volume of corrosion products may be insufficient to cause surface cracking, but it is possible that severe loss of steel cross-section may occur with very little prior warning from visible indications [7].

Stress corrosion can occur in prestressing steel. It is a highly localized corrosion that can lead to cracking of the prestressing steel due to the high stress levels present. The formation of a micro pit occurs in the tendon, and the tip of the pit is subjected to highly concentrated stress. The micro-pit is also undergoing dissolution as the active anode in a localized corrosion process. The combination of stress and rapid corrosion can initiate a crack that propagates rapidly leading to brittle fracture of the tendon [7].

2.3 DEICING SALTS & AND CORROSION DAMAGE

Corrosion can affect every element of a concrete bridge. In coastal regions, most bridge elements are affected more or less evenly by general exposure to chloride-laden air. Bridges over seawater are more affected on their undersides (e.g. deck bottom). Bridges in northern climates are affected differently. Bridge deck reinforcements are susceptible to corrosion because deicing salts are applied on the roadway surface in the winter. Girders experience corrosion mainly at their end regions due to salt-water leaking through failed expansion joints. Piers can be affected along much of their height when they are exposed to salt spray from vehicles, including snowplows, which travel under the bridge. Pier caps and pedestals are also affected by salt-water intrusion through leaky expansion joints.

The use of deicing salts in northern climates is not likely to be discontinued. The use of deicing salts has actually increased in the 1990's after a period of leveling off in the 1980's [54]. Some examples of road salt alternatives include calcium magnesium acetate (CMA) and potassium acetate (KA) [27]. While both CMA and KA appear to be viable road salt alternatives, the high cost of the material and equipment and facility modifications prevent widespread acceptance of these materials.

CMA acts more slowly and is less effective than salt in cold conditions. In general, nearly all studies of CMA rated the substance as an acceptable deicer but not as effective or consistent as salt when applied in equal amounts [27]. In 1991, the National Research Council (NRC) of Canada examined CMA as an alternative to road salt in deicing operations [27]. The study concluded that CMA is relatively harmless to plants and animals, noncorrosive to metals, and nondestructive to concrete and other highway materials. Because of its low density and small

particle size, CMA may be dusty during handling and storage and may blow off roadways after application. In addition, when exposed to moisture, CMA can clog spreading equipment. The calculated ratio of CMA to salt for comparable ice melting is 1.7 to 1 [27]. A study conducted by the Wisconsin Department of Transportation in 1987 [27] reported application rates of CMA 1.2-1.6 times greater than salt. The average 1991 cost of salt was approximately \$30/ton; whereas the cost of CMA was estimated to be between \$500 and \$700/ton [27]. Conversion to CMA would also incur additional costs associated with the modification of storage, handling, equipment and spreading operations.

Potassium acetate (KA) is often used as a base for commercial chloride-free liquid deicer formulations [27]. Its advantages include low corrosion, relatively high performance, and low environmental impact. Less research has been conducted on the application and effectiveness of KA. However, some studies have concluded that the substance has minimal impacts on human health and the groundwater supply [27]. The average 1991 cost of KA was \$700-\$800/ton [27].

The consequences of reinforcement corrosion include cracking and spalling of the concrete. Spalling concrete can be a safety issue for vehicles passing nearby as well as permitting or accelerating further deterioration. Spalled concrete also allows chloride-laden water to reach the reinforcement resulting in more corrosion. Also, as the reinforcement corrodes, the effective cross-sectional area of the steel is reduced, resulting in a decrease of structural strength. Therefore, the overall strength and stiffness of the bridge element is reduced.

2.4 CORROSION REPAIR METHODS

Traditional methods of repairing concrete bridges with corroded reinforcement fall into two general categories: (1) non-electrical (conventional) methods and (2) electrical methods.

Conventional methods include patching, sealers and coatings, overlays, or combinations of these. Surface treatments, such as sealers, coatings and overlays, prevent the passage of potentially deleterious substances and subsequently may slow the deterioration process. The primarily employed electrical method is cathodic protection. Cathodic protection can reduce corrosion rates if the corroding element can be shifted to a cathodic condition through addition of a sacrificial anode with or without an externally applied potential (i.e. impressed current). An additional electrical method, akin to cathodic protection, is chloride extraction. The process involves the application of an external current (much higher than in cathodic protection), which causes the chloride ions to move away from the reinforcement.

2.4.1 CONVENTIONAL NON-ELECTRICAL METHODS

Conventional repair methods are classified into the following broad categories: (1) patches, (2) overlays, (3) sealers and coatings, and (4) crack injection. Each of these methods is employed to repair damaged concrete and to protect from further corrosion damage.

2.4.1.1 Patching

Patching involves removing the concrete area around the damaged region, typically with a chipping hammer, jackhammer or by water blasting. Any exposed reinforcement is cleaned and possibly treated with a corrosion inhibitor. The patch material is then placed inside

formwork or by troweling [15]. Table 1 summarizes the expected life and costs associated with the two common patching options [54].

Table 1. Cost (adjusted to 1998) and Life Expectancy for Patching Options [46, 54]

Type of Maintenance	Average Cost (\$/m ²)	Range of Costs (\$/m ²)	Average Expected Life (years)	Range of Expected Life (years)
Bituminous Concrete Patch	\$90	\$39 to \$141	1	1 to 3
Portland Cement Concrete Patch	\$395	\$322 to \$469	7	4 to 10

Commonly used classes of patch materials include: (1) Portland-cement concrete, (2) hydraulic cement concrete [40], (3) polymer based (e.g. epoxy) patches, and (4) bituminous concrete patches. Portland-cement patches are the most commonly used, and construction workers are typically familiar with the installation techniques. Hydraulic (fast-setting) cement concrete materials are similar to regular concrete. They are generally self-leveling, do not require mechanical vibration, and are more stable at higher temperatures than cementitious materials. Polyurethanes and epoxies are relatively new patch materials. Proportioning and mixing are critical to material performance. Also, because of their relatively low viscosity, they are more difficult to place on vertical surfaces. Bituminous (or asphaltic) patches are commonly used to provide temporary riding surfaces on bridge decks in a rapid manner [17]. However, these patches have shown to have a service life of only 1 year, and should therefore be replaced with more durable patch material [36].

Patch repairs sometimes require partial or complete disruption of traffic because of the need for shoring the member under repair, partial removal of the contaminated concrete from

damaged regions, cleaning of the corroded reinforcement, and placing the patch material. This is a labor-intensive process, yet some argue that patch repairs are not durable [39]. Patch treatments can mend spalls, but typically do not retard chloride-induced corrosion. In such cases, this type of repair will typically fail prematurely since no measures are taken to mitigate the primary source of deterioration. In addition, since the newly placed concrete consists of minimal to no concentration of chlorides, a reverse chloride gradient is created between the patch repair and the existing concrete [39]. The result is the failure of the patch and a need for subsequent repairs. This method, however, is generally successful when the source of damage is related to accidental or load-induced causes. The life of most patch repairs is limited to a maximum of 10 to 15 years.

A study conducted by Patel et al [40] as part of a 1990 Strategic Highway Research Program project (SHRP H-106, Innovative Materials Development and Testing) evaluated the performance of six rapid setting concretes, a polymer modified concrete, a polyurethane, an epoxy, an epoxy-urethane, and two bituminous cold mixes used for repairing partial depth spalls. In conjunction with the evaluation of the repair materials, five patching procedures varying in the methods of concrete removal and surface preparation were studied. These repair methods included saw and patch, mill and patch, waterblast and patch, jackhammer and patch, and adverse condition clean and patch. The “adverse conditions” involved installing the material when the temperature was below 40 °F and lightly spraying the concrete substrate with water.

The study concluded that many of the patch materials were sensitive to water content and placement temperatures. Installation directions must be followed carefully, and appropriate

product precautions, such as using ice water, placing at night, and storing the material in the shade, should be followed when extremely hot temperatures are encountered during placement. Care and understanding must be maintained to complete the repair. Proper surface preparation was crucial in all cases. In addition, the study found that a carbide tipped milling machine might be economical for removing deteriorated concrete when a large area requires repair. However, proper alignment of the milling head required considerable time and additional labor. This method might also pose a traffic hazard due to encroachment into the adjacent traffic lane. The study also found that the high-pressure (30,000 psi) water-blasting machine was not effective. Many equipment failures and an extremely slow concrete removal rate were observed in this particular study.

The patch materials were installed in 1991 from April to July at four test sites in four climatic regions in the United States. In addition, laboratory tests were conducted to identify correlations with field performance. The materials were evaluated on a periodic basis. The study reports findings 1 to 3 days after installation as well as one and three month's results. Figure 4 illustrates the repair performance after three months of evaluation. The percent of patches indicate the fraction of patches experiencing transverse cracking, wear, debonding, or failure. The author considered a patch repair failed when it could no longer carry traffic safely.

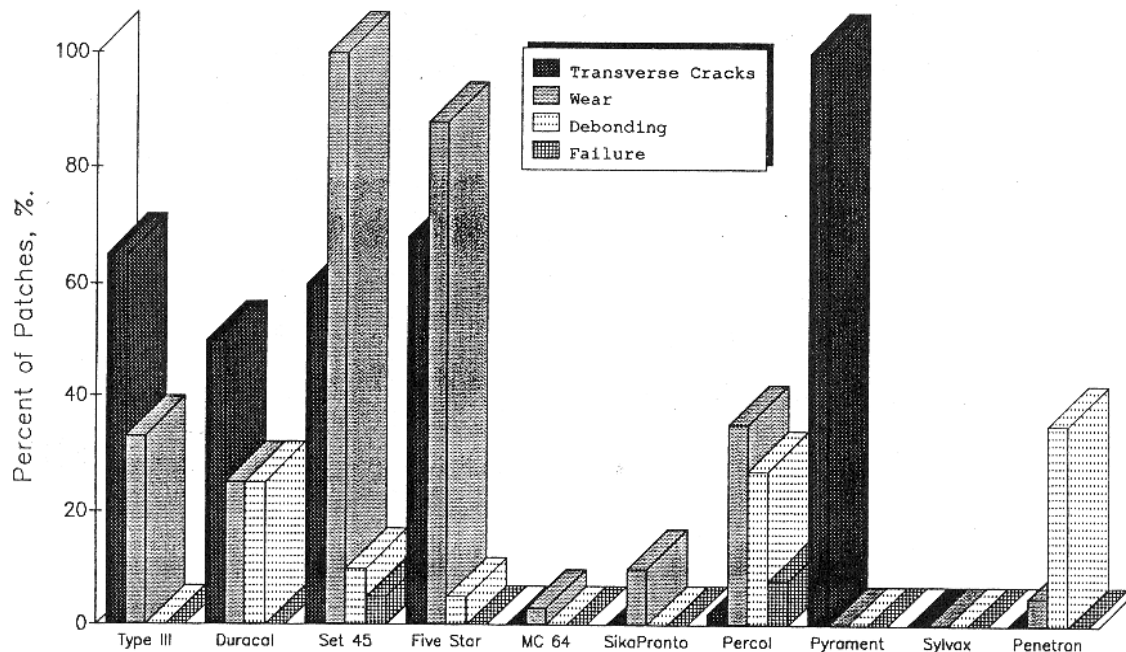


Figure 4. Distress Summary [1]

The report summarized the following observations about the repair materials:

- *Type III cement concrete* is a common patch material and hence the construction crews employed in that study were familiar with the placing, compacting, finishing, and curing techniques. The mix was found workable when the air temperatures were below 80° F, but the mix was found stiff and difficult to work at high temperatures.
- *Pyrament 505* (hydraulic cement concrete) is similar to install and finish as regular concrete and was found to be easy to mix, place, and finish under ambient temperatures. This product took more time for mixing than the other cementitious products evaluated and its workability under cold and wet conditions was more difficult.

- *Percol FL* (polyurethane) required proper equipment and a qualified technician. It was considered critical that clean, oven-dried aggregate be used with this product. Even a small amount of dust or moisture may cause poor bonding or bubbling. This product reportedly had low viscosity and therefore was difficult to place on slopes and grades.
- *MC 64* (epoxy) patches were relatively unfamiliar to the construction workers employed in the study. The proportioning and mixing was considered critical to material performance. Both the mix and finishing required a lot of personnel and time. This product also had low viscosity and was reportedly difficult to place on slopes and grades.

Research conducted for the Federal Highway Administration (FHWA) [52] included a laboratory study into the corrosion of prestressed concrete highway bridge elements and conventional repair methods used for these structures. Test specimens were precorroded through application of anodic current while exposed to chloride solutions. In order to study conventional concrete repairs, it was necessary to remove concrete from preselected areas and replace the chloride contaminated/deteriorated concrete with repair materials. Materials evaluated included conventional portland cement concrete, latex-modified fiber-reinforced patching mortar, and silica fume concrete containing either organic or inorganic corrosion inhibitors.

All specimens were exposed for approximately 200 weeks to a 15% solution of sodium chloride after repair. In many of the specimens, significant deterioration of the coatings applied to the prestressing strands and reinforcing steel had occurred over the four years of severe exposure. The distress was greater for the steel coated with liquid epoxy coating than the steel coated with a zinc-rich product. Typically, there was more disruption of coating and

corrosion of base steel in regions where latex-modified mortar had been used as repair material than where conventional concrete or silica fume concretes were used. Corrosion was observed in repair areas where chloride contents were below commonly accepted threshold levels. When tendon bundles were cut and pulled apart, corrosion was observed on the interior surfaces of the individual strands, indicating that chloride ions had penetrated prior to the repairs. These ions then become available to foster additional corrosion over time after repairs have been applied. At repair area edges, steel coating failure and corrosion were, in general, greater than in the bulk of the repair, and testing demonstrated that chloride ions moved laterally into the concrete, raising the concentration at patch edges. Although only two specimens were available for comparison, an inorganic corrosion inhibitor appeared to be more effective in reducing the extent of corrosion than an organic-based product.

The study concluded that the patch repair systems did not offer long-term protection to rehabilitated prestressed concrete members. Even in low permeability patches (such as those based on silica fume concrete) chloride ions may penetrate vertically from the surface of the members and laterally from the adjoining un-repaired concrete. Field applied steel coatings lost their effectiveness overtime and deteriorated, exposing the underlying steel to corrosive agents. The study recommended that periodic repair and reapplication of protective systems might be necessary to maintain structures. Where periodic repairs are difficult to carry out, the study suggests complete replacement of distressed members may be a long-term cost-effective alternative.

A study conducted in 1993 by Sprinkel et al [45] for the Strategic Highway Research Program evaluated various rapid repair methods including the performance characteristics of some

patch repair materials. The report states that patching methods can mend corrosion-induced spalls, but typically do not retard chloride-induced corrosion because not all concrete containing chlorides is removed. The research determined that the corrosion rates are high at the perimeter of the patch and is independent of the type of patch material used.

2.4.1.2 Overlays

Bridge deck overlays are primarily used to improve durability and service life of bridge decks. They can restore the quality of the deck surface and increase the effective cover for the reinforcement. An overlay must have a long-term stable bond with the repaired deck and sufficient resistance to environmental conditions such as vehicle traffic and chloride-laden water. Overlays are most effective when used in conjunction with a system that protects against further corrosion, such as corrosion inhibitors or cathodic protection. However, overlays do not address the presence of chlorides. Overlays typically extend the life of a deck 6 to 10 years [22].

A study conducted by the Michigan State University researched factors affecting the service life of corrosion damaged reinforced concrete bridge superstructure elements [36]. The study concluded that the amount and degree of contaminated concrete left in place influence the effectiveness of an overlay. Since the extent of surface damage primarily influences the decision to overlay a bridge deck, the amount of contaminated concrete left in place is similar for various environmental exposure conditions. Thus, if the original base concrete is not replaced or rehabilitated, the service life of overlays can be similar for all environmental exposure conditions.

Overlay permeability is an important material characteristic. High permeability allows moisture to penetrate through the overlay and into the concrete below. Permeability depends on the porosity of the overlay and the presence of cracks. Therefore, overlay cracking should be minimized, whether from shrinkage, thermal stresses, or fatigue, to prevent deterioration of the overlay. Considering the type and thickness of the material can lessen overlay cracking.

Table 2 summarizes the service life and costs associated with different overlay and patching options [54].

Table 2. Cost (adjusted to 1998) and Life Expectancy for Overlay Options [46, 54]

Type of Maintenance	Average Cost (\$/m ²)	Range of Costs (\$/m ²)	Average Time until Maintenance (years)	Average Expected Life (years)	Range of Expected Life (years)	Typical Thickness (in)
Portland Cement Concrete Overlay*	\$170	\$151 to \$187	8.3	18.5	14 to 23	≥1.3-2.0
Bituminous Concrete with Membrane	\$58	\$30 to \$86	5.1	10	4.5 to 15	≥1.6
Polymer Overlay**	\$98	\$14 to \$182	6.4	10	6 to 25	≥0.3-0.5

*Includes latex-modified concrete (LMC).

**Polymer Overlays include: epoxy, epoxy urethane, methacrylate, polyester styrene, & polyurethane

Three common overlays include: latex-modified concrete (LMC), low-slump dense concrete (LSDC), and silica-fume concrete (SFC). Traditionally, LMC overlays constitute more than 90% of the overlays used for rehabilitation applications [54]. The LMC and SFC overlays are generally less permeable than dense concrete and are stronger, allowing a reduction in required thickness. However, increased thickness can be an advantage in protecting the underlying deck if the overlay cracks.

Detwiler et al [13] reported on the overlay of the IL 4 Bridge over Interstate 55 near Staunton, IL. In October 1986, the southbound lane was repaired using a standard LSDC overlay; and the northbound lane was repaired in March 1987 using a SFC overlay. The bridge provided an opportunity to compare the performance of the overlays using the same contractor to install the repairs and be exposed to the same environmental conditions. The overlay repairs were evaluated in July 1995. According to field survey and petrographic examinations, both the LSDC and the SFC overlay repairs were originally of high quality. Both performed well under the exposure conditions. The silica fume concrete appeared to provide better protection against the ingress of chloride ions. These results were consistent with the chloride ion profiles, which generally indicated that the chloride ion concentration of the silica fume concrete to be lower than for the low slump dense concrete at a given distance from the surface.

A study conducted by Sprinkel et al [45, 46] evaluated the performance of polymer overlays and concrete overlays. The study concluded that polymer overlays have a useful service life of 10 to 25 years when applied as a protection or rehabilitation treatment. Application of a multiple layer epoxy, a multiple-layer epoxy-urethane, a premixed polyester styrene with a methacrylate primer, or a methacrylate slurry were determined to be the “best-proven” overlay treatments. The research also concluded that high-early-strength hydraulic cement concrete overlays have tremendous potential, but considerable developmental work with the materials and equipment would be needed to overcome problems with installation time. High-early-strength portland cement concrete overlays such as those prepared with 7% silica fume or 15% latex and Type III cement reportedly had a potential service life of 25 years and could perform as well as conventional overlays with quicker installation and curing times.

A relatively new overlay material is conductive-concrete overlay. When connected to a power source, these overlays can generate enough heat through electrical resistance to prevent ice formation on bridge decks, or melt ice after it forms [53]. Conductive concrete is a cementitious mixture containing electrically conductive components that give it a stable and high electrical conductivity. The University of Nebraska and Nebraska Department of Roads have developed a conductive overlay that has “excellent workability and surface finishability” [53]. The studies indicated that conductive concrete containing 20% steel shavings and 1.5% steel fibers had the optimum resistivity and workability characteristics. Studies also indicated that anti-icing was more cost effective and energy efficient than deicing. The average energy cost per unit surface area is about \$0.074/ft² per storm (for Omaha, Nebraska). The material cost of conductive concrete is about \$270/m³ compared to \$51.3/m³ for conventional concrete. Although the cost of conductive concrete is higher, these overlays should be considered a heating element rather than repair materials [53]. In order to evaluate the durability of conductive concrete overlays under traffic loads, an overlay patch was placed on a bridge deck of I-480 near the Nebraska/Iowa border in December of 1999 [53]. Visual inspections were conducted every 6 months, and indicated that there was no fiber exposure or any sign of corrosion, but some reflective cracking did develop.

2.4.1.3 Surface Treatments

The concrete is relatively porous and will absorb moisture. Absorbed moisture can lead to surface scaling and spalling when subjected to freeze-thaw cycles. If the water was contaminated with chlorides, steel corrosion would occur.

Blocking the ingress of water and other deleterious substances could reduce the natural permeability of concrete. In existing concrete, sealers and coatings could be used to form a seal that reduces the permeability of concrete. Two types of common surface treatments include coatings and penetrating sealers. The classification is based on the behavior of the treatment. The treatments provide a non-penetrating film, penetrate into concrete pores, or have intermediate behavior (Figure 5). As a preventative maintenance strategy, coatings and sealers offer significant long-term benefit when applied early on, especially in environments exposed to chlorides.

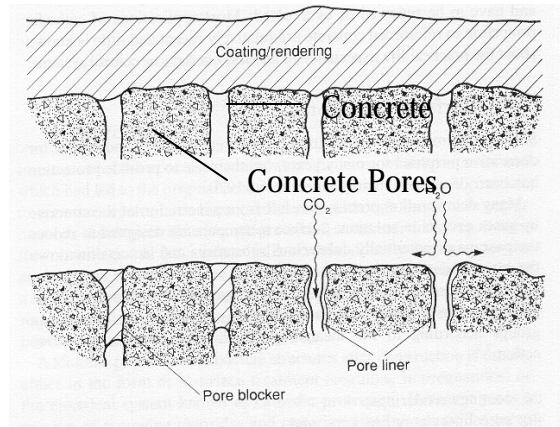


Figure 5. Coatings (top) versus Sealers (bottom) [26]

2.4.1.3a Coatings

Coatings form an impervious film that bridge over pores and provide an external physical barrier, which protects by slowing down the penetration of liquids and gases. They are designed to control water absorption, vapor transmission, and diffusion of aggressive liquids and gases through the concrete surfaces. Coatings are normally applied by brushing, rolling, or spraying the material onto the surface and are available in a variety of colors.

Two commonly employed coatings include cementitious and polymer coating systems.

Cementitious coatings allow moisture to escape without debonding or blistering. However, since they do not possess elastomeric properties, they cannot bridge active cracks [8]. Polymer systems consist of epoxies, acrylics, or polyurethanes combined with filler, which provides bulk and thickness. Polymer coatings are hard and durable, but are impervious to vapor transmission. These coatings have blistered and peeled under high vapor pressure [8]. Mallett [36] reports that coatings can be expected to last 10 years and that some appear to be performing well after 30 years.

Bijen [31] summarizes the characteristics and performance of some common coatings. He reports the following:

- Epoxy coatings provide good adhesion to concrete, exhibit minor shrinkage, and are resistant to light chemical attack.
- Polyurethanes will adhere to dry concrete, are almost shrinkage free, are resistant to light chemical attack, but not to highly alkaline conditions.
- Acrylics display good adherence to concrete and good resistance to alkali, oxidation, and weathering.

Ibrahim et al [23] studied the effectiveness of concrete surface treatments including sealers and coatings. They evaluated several penetrating sealers (detailed in section 2.4.1.3b) and a two-component acrylic coating. The coating was found to be the most effective of the materials investigated in minimizing damage due to sulfate attack after 6 months of sulfate exposure. In addition it was determined to be effective in reducing the ingress of carbon dioxide. The

coating also exhibited considerably lower chloride concentration in comparison to untreated specimens.

2.4.1.3b Sealers

Sealing existing concrete surfaces reduces the permeability of concrete, which can be improved up to one order of magnitude [9]. This is comparable to using silica fume or latex admixtures to reduce the permeability of new concrete [9]. Experiences in Canada indicate that, “if deterioration has not already progressed too far, maintaining the permeability of existing, exposed concrete at the levels obtainable with correctly applied sealers can reduce the rate of deterioration and result in a reasonably standard service life for exposed concrete” [9].

Penetrating sealers are low viscosity liquids that are capable of penetrating into concrete surface pores filling the cracks and voids. Two types of penetrating sealers include pore liners and pore blockers. Pore liners line the concrete pores and enable the concrete surface to become water repellant. Pore blockers penetrate into the pores and then react with concrete constituents. The resulting products are insoluble and hence, block the concrete pores [26].

Boiled linseed oil is one of the oldest materials used to seal concrete surfaces. It is low in cost, but in cases where it is exposed to traffic abrasion it must be frequently reapplied to maintain protection [8]. Silane sealers penetrate about $\frac{1}{2}$ inch into the concrete and react chemically with concrete to form a layer that resists water and chloride penetration [8]. Siloxane is very similar to silane although not as effective in reducing water and chloride penetration [8]. Both silane and siloxane are sealers permeable to water vapor, which allows the concrete to dry out. In addition, both substances do not color the concrete.

Sodium silicate sealers penetrate concrete and react with calcium compounds to form insoluble calcium silicate, reducing the permeability. Penetrating epoxy sealers use a chemical reaction between the resin and hardener to create a protective film [8]. They permit some moisture vapor transmission, but normally allow less transmission than materials that do not develop a protective film [8].

A study conducted by Whiting et al [51] surveyed highway agencies in the United States and Canada regarding the use of penetrating sealers. The study did not specifically mention the type of sealer that was presented on the questionnaire. Of the agencies surveyed, 46 U.S. and 9 Canadian agencies employed sealers. The most widely used application of penetrating sealers was reported to be on concrete bridge decks. Only about 30% of the respondents were utilizing penetrating sealers in superstructure elements other than decks. The study stated that the respondents noted a variety of problems with the application of penetrating sealers to existing structures. Some of these concerns included the following: drift and evaporation in hot and windy conditions, difficulty in obtaining specified coverage, slippery surfaces, runoff during application, discoloration of concrete, and little or no apparent penetration. The respondents also stated that the performance of the sealers was less than desired. Some indicated that many penetrating sealers were ineffective (or at least not as effective as claimed) in reducing chloride ion infiltration. Other performance problems included: reduction of skid resistance, failure to improve freeze-thaw and scaling resistance, and failure to halt corrosion of reinforcing steel.

Sprinkel et al [45, 46] studied the performance characteristics of sealers as well as overlays and patch materials (section 2.4.1.2). The study did not specifically indicate the type of sealer that

was evaluated. The investigation concluded that sealers could reduce the infiltration of chloride ions for 5 to 10 years and therefore extend the time until sufficient chloride ions reach the reinforcing steel to initiate corrosion. To ensure adequate skid resistance, sealers should be applied to decks with tined or grooved surfaces. The investigation found that protection provided by sealers varied with tests, exhibiting 0 to 50%, with an average of 32%, reduction in permeability. On the basis of life cycle cost analysis, the most cost effective protection system was determined to be the application of a penetrating sealer.

In contrast to coatings, penetrating sealers allow the concrete to breath since the pores are exposed to the atmosphere. This permits the concrete to dry-out, and with the moisture intake reduction the possibility of corrosion may be lessened. Since most sealers are clear in color, the color of concrete will generally not be affected when applied. In addition, since penetrating sealers are capable of infiltrating well into the surface, they are less affected by environmental weathering. This can lead to a longer service life when compared to coatings [26]. Bruner [8] compiled a table (reproduced in Table 3), which rates the performance of various sealers and coatings based on several criteria.

Table 4 summarizes the results of a study conducted by Ibrahim, Al-Gahtani, and Dakhil [23], which evaluated the effectiveness of sealers and coatings.

Table 3. Concrete Surface Treatment Selection Guide [8]

Material Property	Boiled Linseed Oil	Silane	Siloxane	Sodium Silicate	Penetrating Epoxy	Cementitious Coating	Epoxy Coating
Ability to Penetrate	A	G	G	G	G	N/A	N/A
Ability to Bridge Cracks	N/A	N/A	N/A	N/A	N/A	P	VP
Ability to Bond to Concrete	N/A	N/A	N/A	N/A	N/A	G	G
Ability to Reduce Permeability	A	G	A	G	G	G	G
Allow Water Vapor Transmission	A	G	G	G	P	A	VP

VG – Very good performance

G – Good performance

A – Average performance

P – Poor performance

VP – Very poor performance

N/A – Not applicable

Table 4. Ranking of Surface Treatments [23, 36]

Sealer/Coating	Environment		
	Sulfate Attack	Carbonation	Chlorides
Control (no sealer)	7	7	7
Sodium Silicate	6	3	6
Silicone Resin Solution	5	5	5
Silane/Siloxane	4	4	4
Silane/Siloxane with an Acrylic Topcoat	1	1	1
Alkyl-Alkoxy Silane	3	6	3
Two Component Acrylic Coating	2	2	2

(Scale from 1 to 7, a rating of 1 implies the best performance)

This study assessed the performance of sodium silicate, a silicone resin solution, silane/siloxane, silane/siloxane with an acrylic topcoat, alkyl-alkoxy silane, and a two-component acrylic coating in preventing concrete deterioration due to sulfate attack, carbonation, and chloride-induced reinforcement corrosion. The study concluded that the

penetrating sealers were not effective in reducing concrete deterioration due to sulfate attack. However, silane/siloxane and silane were partly effective in decreasing sulfate attack. Silane/siloxane with an acrylic topcoat was reported to be the most effective in reducing sulfate attack. The investigation also determined that none of the penetrating sealers were totally effective in preventing carbonation of concrete. In addition, the sealers did not perform as well as the coatings in reducing chloride diffusion. However, the coatings (both the silane/siloxane with acrylic topcoat and acrylic coating) were found to be the most effective in preventing carbonation, decreasing chloride diffusion, and reducing reinforcement corrosion. The performance of the surface treatments investigated in the study can be expressed in the following order: silane/siloxane with an acrylic topcoat > acrylic coating > silane > silane/siloxane > silicone resin solution > sodium silicate.

A study conducted for the Transportation Research Board in 1981 [41] researched the protection of concrete bridges against chloride penetration by various surface treatments (coatings & sealers) representative of all of the chemical types commonly used. Initially, 21 surface treatments including epoxies, methacrylate, urethanes, butadienes and a silane were subjected to preliminary screening tests. Based on the initial screening program, five products with low water absorption, low chloride ion uptake and good water vapor transmission characteristics were chosen for further testing. The five materials chosen were an epoxy, a methyl methacrylate, moisture cured urethane, a silane and polyisobutyl methacrylate. These materials were subjected to further testing to determine the effects of moisture condition of the substrate, coverage rate and different environmental conditions on the ability to protect against chloride ion intrusion. The five treatments reduced the chloride ion contents by 79 to 97% compared to the uncoated specimens [31]. The study concluded that the epoxy, methyl

methacrylate and the silane were capable of providing added protection to concrete bridge surfaces to reduce intrusion of salt laden water.

Although in theory surface treatments can provide adequate protection against the initiation of corrosion, the reality of their effectiveness is quite different. These materials may inhibit the penetration of deleterious substances, but they do not mitigate the effects of the chlorides that are already present. The generally expected service life of surface treatments is approximately 5 years [22].

A study conducted by the Federal Highway Administration [52] evaluated various corrosion repair techniques and protection systems for prestressed concrete elements. The prestressed concrete specimens were subjected to accelerated corrosive environments to induce corrosion in the steel. Penetrating sealers and coatings were applied to a set of specimens to study their effectiveness. This study concluded that the surface treatments were of limited effectiveness when applied to specimens subjected to active corrosion. In most cases, chlorides continued to penetrate into the concrete, though at a reduced rate. “While measurements indicated that corrosion activity was initially reduced after the application of the surface treatment, long-term trends suggest that over time corrosion activity may slowly increase back towards the initial levels [52].” The study also concluded that surface treatments applied to new structures would reduce, but not completely eliminate the ingress of deleterious substances. If low quality repair materials are used or incorrect construction procedures are employed, corrosion may still occur resulting in the cracking and spalling of the structure. However, in general, application of surface treatments in new construction significantly improves its long-term effectiveness, especially in chloride environments.

2.4.1.4 Crack Injection

Crack injection involves infusing cracks and other voids with a low viscosity adhesive material (resin). This process glues the concrete together and restores some of the original strength. A protective overlay or surface treatment can then be applied to the surface to prevent moisture penetration and continued deterioration [4]. It is essential to determine the cause of cracking and choose a resin with proper characteristics to ensure the effectiveness of the repair [26].

Crack injection is not applicable to cracks caused by reinforcement corrosion, or if a continuing process is responsible for their generation [26]. Cracks repaired by injection normally perform well if the cracks are dormant, but have not performed as well for active, moving cracks. The method is typically used and successful for hairline cracking and delaminations found on fewer than 30% of the deck area [2]. Sprinkel [46] reports that crack repair has an average service life of 10 to 20 years.

Resins normally consist of two components, an active ingredient and hardener. If large cracks or voids are to be treated, inert filler is also included. The components must be carefully and thoroughly mixed to obtain a final product with the desired properties [26]. The surface of the crack is cleaned and then sealed using polyester putty or other suitable material. One method of crack injection involves introducing the resin under pressure at the first port with the other ports open (see Figure 6).

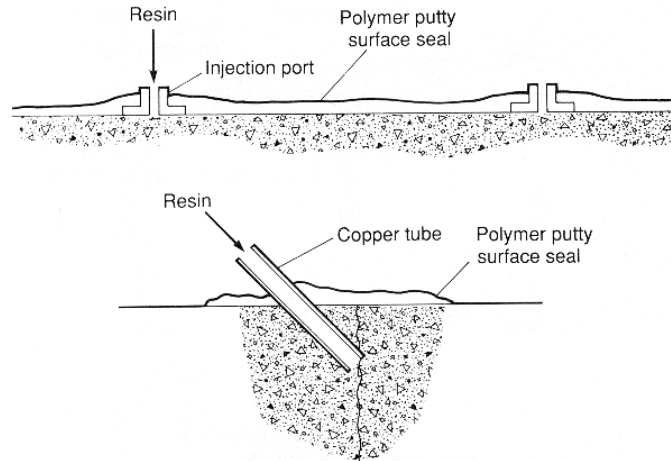


Figure 6. Crack Injection (Under Positive Pressure) [26]

Injection continues until the resin appears at the second port. The first port is sealed off and the process is repeated for subsequent ports. During injection, the region is monitored for signs of resin leaks from interconnected cracks or voids. When the process is complete, the ports are removed [26].

ACI Committee 244 [1] reports that epoxy injection has been successfully used in the repair of cracks in concrete structures. However, the document concludes that unless the cause of cracking has been corrected, the cracking will probably reoccur near the original crack. In addition, the report states that this technique is normally not applicable if the cracks are actively leaking and cannot be dried out. Wet cracks can be injected using moisture tolerant materials, but contaminants in the crack can reduce the effectiveness of the resin to structurally repair the crack.

In 1990, Calder [31] studied the protection afforded by crack injection to reinforced concrete slabs. The materials investigated included: epoxy resin, polyester resin, methyl methacrylate (MMA) resin and liquid silicate solution. Comparisons with unrepaired slabs after ponding

with salt water for a period of three years led to the following penetration ranking: epoxy (best) > polyester > MMA > silicate (least good). These repairs prevented preferential access of chlorides into the cracks but did not prevent penetration of chloride ions from the surface. The investigation found that 80% of the corrosion was located at the cracks. The repairs reduced the total number of corrosion sites by half, but had little effect on the number of sites experiencing some section loss. There was little performance difference between the resins in improving the concrete durability, but the silicate solution was determined to be ineffective. However, each of the resins reduced the carbonation depth in comparison to the untreated specimens.

2.4.2 ELECTRICAL METHODS

Electrical repair methods can be classified into the following categories: (1) cathodic protection and (2) chloride extraction. Each of these methods is employed to arrest corrosion and to prevent further corrosion damage.

2.4.2.1 Cathodic Protection

Typical cathodic protection systems include the impressed current system and the sacrificial anode system. The impressed current system employs an external direct current supply. Corrosion is arrested by subjecting the reinforcement to a small direct current to prevent it from reaching an electrical potential that could cause corrosion. The sacrificial anode system uses an external anode, a metal higher in the electrochemical series (i.e. zinc), which corrodes in the process of providing protection. Sacrificial anode systems are simpler than impressed current systems. Sacrificial anode systems can use recycled materials, which can make them

less expensive than impressed current systems. However, impressed current systems are usually employed because of its greater current range, ease of adjustment, and its longer service life [42].

Cathodic protection, if applied properly, can arrest steel corrosion in concrete. In 1982, a report by the United States Federal Highway Administration [31] stated, “The only rehabilitation technique that has proven to stop corrosion in salt-contaminated bridges regardless of chloride content is cathodic protection”. Since corrosion is an electrochemical process, controlling the flow of an externally applied electrical current (impressed current method) can control corrosion. By applying an external potential, the corrosion rate is reduced by shifting the embedded steel to an artificially cathodic condition. The reinforcement is made cathodic relative to an anode located at or near the concrete surface. Cathodic protection eliminates electrolytic attack of steel and repels dissolved chlorides. If substantial corrosion exists, then cathodic protection could offer a more economical solution than extensive patching repairs [42]. If there is no loss of structural integrity, only repairs to spalled and delaminated concrete are required. There would be no need to remove large volumes of contaminated concrete. Cathodic protection has a high initial cost, but can extend the service life 20 to 30 years [22]. Presently cathodic protection remains an under-used technology for corrosion protection [54]. Table 5 summarizes the cost and life expectancy of several cathodic protection systems.

Table 5. Summary of Costs and Life Expectancy for Cathodic Protection Systems [54]

Type of Maintenance	Average Cost (\$/m ²)	Range of Costs (\$/m ²)	Average Expected Life (Years)	Range of Expected Life (years)
Impressed-Current (deck)	\$114	\$92 to \$137	35	15 to 35
Impressed-Current (substructure)	\$143	\$76 to \$211	20	5 to 35
Sacrificial Anode (substructure)	\$118	\$108 to \$129	15	10 to 20

Three types of anodes used in impressed current systems include conductive mastic anode, conductive rubber anode, and titanium mesh anode. In the mid-1980's, Florida DOT began employing cathodic protection featuring conductive mastic anodes on its coastal bridges. Mastic paint was initially used, which included carbon to enhance its conductivity, on the regions requiring protection. A rectifier was installed at a central location on the bridge and wires were routed to the protected areas. The DOT experienced favorable results on the beams and decks, but encountered problems with the piles. Water from the high tides impaired the bond between the mastic and the piles and thus the current was poorly distributed to these regions [28].

The conductive rubber anode was developed to address this problem. The rubber anode can be in direct contact with water and continue to distribute current uniformly. The anode is a rubber mat that includes a large amount of carbon. The rubber mats are bonded to the concrete areas to be protected and then connected by wires to a rectifier. This system has performed well and has an expected service life in excess of 20 years [28].

When concrete is still in fairly good condition and only requires some patch repairs, the conductive mastic and rubber anode systems are practical and effective. However, if concrete has severe cracks and spalls and requires more extensive repairs, the titanium mesh system is more practical. Titanium mesh is fastened directly to the concrete element after all the loose and damaged concrete has been removed. The mesh is then embedded in a 2-inch thick gunite (shotcrete) coating. This system has experienced problems when the coating is in direct contact with water. If a member is determined to be structurally deficient, a reinforced concrete structural jacket can be used in conjunction with the titanium mesh. Forms are placed and the concrete is cast around the deteriorated member. The anode can be connected to both the existing reinforcement and any new reinforcement. The mesh-jacket system has proven to be effective in controlling corrosion [28].

The previous impressed current systems all used an external power source to provide the current to the system. The sacrificial anode system provides current by using a metal that is higher in electro-chemical potential than steel, with zinc being the most commonly employed. The anode can be applied either as a coating or sheets. The coating is sprayed on cleaned concrete and exposed reinforcement. The zinc coating typically has a service life on the order of 10 years, at which time it can be re-applied [26]. This system is recommended for applications not in direct contact with water since this accelerates the consumption rate of the anode, and significantly decreases the anode service life [28]. Another system employed uses zinc mesh sheets that are mechanically fastened to concrete. This system is typically used on bridge piles that are in direct contact with water [28].

There are several issues that must be considered when using cathodic protection to protect against corrosion. Selection of the proper anode for the application is critical, failure of the anode leads to failure of the system. In order for cathodic protection to be effective, the steel must be electrically continuous. Ensuring continuity after construction can be an expensive and difficult procedure [42]. In addition, the cathodic protection system itself, which includes an anode, power supply, and monitoring equipment, is costly in comparison to conventional repairs and requires constant monitoring. When an impressed current system is used there are difficulties in determining the correct applied potential and applying it uniformly to the system [3]. The principal concerns include the degradation of the steel/concrete bond, the hydrogen embrittlement of the steel and the alkali-aggregate reaction in the interfacial region [38]. In view of these issues, there is a need to monitor cathodic protection systems continuously to assure that they provide effective protection without detrimental side effects.

Degradation of the steel-concrete bond, associated with the softening of the cement matrix in contact with the metal has been reported in several studies that involved the application of high current densities for prolonged periods [38]. However, at the lower current densities normally required for cathodic protection, the bond strength is normally sufficient to minimize the concern for the structural integrity of the structure [14].

Hydrogen embrittlement is a significant concern for cathodic protected prestressed steels. Hydrogen embrittlement occurs when steel is under high stress and a cathodic reaction is occurring simultaneously at its surface. The cathodic reaction evolves hydrogen atoms at the steel surface. The hydrogen atoms can diffuse and dissolve into the most highly stressed zones of the steel. The effect of hydrogen embrittlement in the stressed zone is to embrittle the

steel, which can lead to the brittle fracture of the tendon [7]. The risk of embrittlement depends on a number of factors, and it appears to be low provided the potential is maintained at a level less negative than -900 mV [38].

Wagner [50] reports on research conducted regarding the use of cathodic protection of highly stressed steel tendons. Their research indicates that hydrogen penetrates steel and causes ductility reduction at potentials equal or more negative than those normally considered for the thermodynamic stability of iron. Their experimental work indicates that even short-duration exposure to cathodic potentials of significant magnitudes can produce hydrogen in the metal. The study also found that cathodic potentials more negative than the hydrogen evolution potential sustained for durations greater than 2 hours will result in a reduction in the dynamic load-carrying capacity of notched steel tendons. However, the results indicated that potential levels more negative than the hydrogen evolution potential would not result in a reduction in the static load-carrying capacity of unnotched prestressing tendons. The research continues to be conducted to determine the effectiveness of cathodic protection in known salt-contaminated full-sized prestressed concrete beams.

A limited number of laboratory studies have indicated a potential problem when cathodic protection is used on reinforced concrete structures constructed with alkali reactive aggregates [14]. If the cathodic current density is uniformly and consistently maintained at a low level, the risk of developing expansive alkali silica reaction (ASR) is reduced [38]. However, if the current distribution to the cathode is not reasonably uniform, the risk of locally induced ASR will be greatly increased. Hence, the European Draft Standard [38] recommends that the risk of ASR be considered.

2.4.2.2 Chloride Extraction

Chloride extraction involves the application of an external current, which causes the chloride ions to migrate away from the reinforcement and generates hydroxyl ions, which increases the alkalinity of the region. Research has shown that after application of this technique, the chloride concentrations are substantially reduced and a corresponding increase in the pH of concrete is observed [26]. Similar to cathodic protection, a distributed anode and overlay is applied to the surface. The overlay normally consists of sprayed cellulose fiber saturated with an alkaline solution [26]. The chloride ions migrate away from the steel and towards the anode and are removed with the overlay. Analogous to cathodic protection, there is no need to remove large regions of chloride contaminated concrete and then replacing it with new material before the application of this technique. However, any loose areas have to be repaired to ensure a continuous medium between the reinforcement and surface anode [26].

A typical chloride extraction system is illustrated in Figure 7. Electrical connections are established with the reinforcement. The temporary anode is installed on wooden battens using plastic plugs and bedded into the fiber layer. Anodes should be easy to bend and shape and are usually in the form of a mesh. Titanium meshes are inert and recyclable. Finally, an additional layer of cellulose fiber is sprayed over the mesh and connections are made to a power source [26].

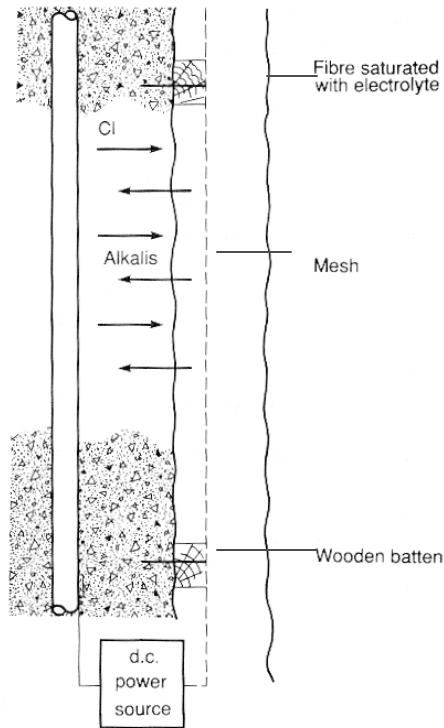


Figure 7. Chloride Extraction and Replenishment of Alkalies [26]

The replenishment of the alkalis is a quicker process than extraction of chlorides. Chloride extraction normally requires three to eight weeks to complete, whereas alkali replenishment requires three to six days to complete [31]. Extraction of chlorides is more difficult because some chloride ions may be bound with reaction products with the cement hydrates. The bound chlorides are in dynamic equilibrium with the chlorides dissolved in the pore water. As the process removes the chlorides in the solution, chlorides from the reaction products replace them. The rate at which chlorides can be extracted is controlled by the decomposition rate of the hydrates [26]. To overcome this effect, current has been applied intermittently to the system.

Bennett et al [6] conducted laboratory tests to determine the feasibility of chloride removal from reinforced concrete bridge components. The studies clearly show chloride extraction to be an effective technique for arresting chloride-induced corrosion of reinforcing steel. After 3 ½ years no specimens showed a tendency to return to a corrosive condition. By contrast, the untreated control slab was badly delaminated and deteriorated. The treatments removed 20-50% of the chloride ions from the concrete, and relocated the remaining chloride ions away from the reinforcing steel. The percentage of chloride removal was dependent on the design of the reinforcement, with regards to spacing and bar placement, the degree of chloride ingress, and the chloride ion distribution.

The study also addressed several concerns, which may arise as a result of the passage of large amounts of current through concrete. The steel/concrete bond strength was measured over a wide range of current and charge. The application of very high current densities resulted in a reduction in bond strength when compared to control specimens. However, the use of lower current densities had no adverse effects. The compressive strength of the concrete was also reduced at high current densities as the specimens experienced a softening of the cement paste around the steel. The possibility of hydrogen embrittlement of the steel was also studied. Although a slight, temporary loss of ductility was observed, the researchers concluded that this loss was not structurally significant. The study also concluded that chloride extraction can cause an increase in the alkali cation concentration in the vicinity of the reinforcing steel and serious damage could occur if the chloride removal process was applied to concrete containing alkali-reactive aggregates. The use of a lithium borate buffer could be used to mitigate this problem.

Bennett et al [5] also reviewed field trials applying the chloride removal process to reinforced concrete bridge components. Four field validation trials were conducted between the fall of 1991 and fall of 1992. Chloride removal was conducted on an Ohio bridge deck, and bridge substructures in Florida, New York and Ontario. Active corrosion was occurring on a substantial portion of each selected structure, and chloride contamination was well above threshold levels. The treatment was applied until a total charge of 60 to 135 A-hr/ft² of concrete was accumulated. The pH of the electrolyte was maintained neutral or basic to prevent etching of the concrete surface and the evolution of chlorine gas. In summary, all four field trials were deemed successful and no detrimental side effects were observed. The report mentions that as of yet chloride extraction cannot be recommended for structures that contain prestressing steel or alkali reactive aggregates.

Manning and Pianca [32] report on the initial evaluation of electrochemical removal of chloride ions from a section of a concrete pier located on the Burlington Skyway. The evaluation included visual examination, corrosion potential, rate-of-corrosion measurements, and petrographic examination and measurement of chloride ion profiles from samples removed from the structure. After 13 months of treatment it was determined that the process was successful in moving chloride ions away from the reinforcing steel and in removing a substantial proportion from the concrete without apparent damage to the concrete. However, the process was unable to remove all of the chloride ions from behind the reinforcing steel. Therefore it is unknown the extent to which the chloride ions will initiate further corrosion in the pier.

A study conducted by the University of Minnesota [10] investigated methods for mitigating corrosion in reinforced concrete structures on the substructure of a bridge in Minneapolis, Minnesota. Several corrosion-damaged columns and pier caps were treated with electrochemical chloride extraction (ECE). Some structures were also wrapped with fiber-reinforced polymer (FRP) sheets or sealed with concrete sealers to prevent future chloride ingress. Embeddable corrosion monitoring equipment (resistivity probe) was installed to evaluate the effectiveness of the ECE treatment. The initial chloride concentration were reduced approximately 50% at each sample depth in each structure. The treatment was most effective near the concrete surface, and the overall effectiveness appeared to depend on the original chloride content (with locations containing high initial chloride concentrations being treated more effectively) and the proximity of the sample to the reinforcing steel. Several locations possessed chloride concentrations in excess of established corrosion thresholds. The study concluded, while the majority of the treated structures can be considered passive, corrosion can potentially reoccur once chloride ions remaining in the concrete migrate back to the reinforcing steel level.

2.5 FIBER REINFORCED POLYMER (FRP)

Due to widespread deterioration of concrete bridges, new materials and protection systems must be investigated to minimize costs and conserve resources. Past research and field studies have proven that patch repairs lack longevity and are susceptible to ongoing deterioration. The limitations imposed by conventional repair materials have led to the investigation of materials that have been widely used in the aerospace, sporting goods, and automotive industries. Fiber-reinforced polymer (FRP) composites are being increasingly considered for

bridge applications due to their high strength-to-weight ratios, their corrosion and fatigue resistance, their ease of transport and handling, and their potential for tailorability. These materials have been implemented in a number of rehabilitation and demonstration projects. The application of FRP composites has been demonstrated to be a promising repair solution for many rehabilitation issues.

Composites are created through the combination of two material phases, one serving as the reinforcement and the other as the matrix. In generic terms, FRP composites are analogous to reinforced concrete. The fiber reinforcement can carry load in pre-designed directions and the resin behaves as a medium to transfer stresses and provide physical protection for the fibers. Common types of fiber used in structural applications include glass, aramid or carbon. Epoxy and polyester are the most common resins. The resulting composite behavior depends primarily on the fiber volume and direction, the mechanical properties of both constituents, and the fabrication procedure. Since composites have variable properties, a wide palette of material choices is available to the designer to fit the specific requirements of the situation.

Unlike metals, composites do not corrode, which makes them attractive in corrosive environments. Composites can be used in new construction as reinforcing bars and grids, or prestressing tendons to eliminate the development of corrosion. Composites have also been studied for their use as surface treatments to provide a barrier to corrosive elements [29, 39].

Several rehabilitation methods have been developed to repair and strengthen concrete structural members. These include the application of externally bonded FRP plates, the use of external or internal FRP prestressing strands, the use of composite wrap to repair corrosion-damaged elements, or even entire replacement bridge decks composed of composite sections.

These structural systems combine the mechanical characteristics of composite fibers, the compressive characteristics of concrete, and the ductility and deformation capacity of steel. In addition, these lightweight materials can reduce the quantities of steel and concrete in the structure, lower material transportation costs, enable quicker construction times, and lower labor costs. Some investigators believe that these cost savings can offset the higher material costs of FRP composites [37].

Recently, the use of fiber-reinforced composites (FRP) to repair damaged girders has been studied. Meier et al [33] studied the effectiveness of FRP plates to strengthen existing girders. Composites offer performance advantages not found in other materials (e.g. steel plates). These advantages include: corrosion resistance, easy to handle, available of endless bobbins therefore no joints are necessary, some do not debond when subjected to compressive stresses, and outstanding fatigue behavior [33].

Tedesco et al [48] performed a comprehensive finite element analysis of a deteriorated reinforced concrete bridge repaired with externally bonded FRP plate. The plates were unidirectional with the fibers oriented parallel to the longitudinal axis of the plate. The FRP plates were bonded to the concrete with readily available structural adhesive. Static and dynamic analyses of the bridge were conducted for conditions both before and after the FRP repairs, with loading by two identical test trucks of known weight and configuration. The results indicated that the bonding of the plates to the bridge girders reduced the average maximum mid-span girder deflections and reinforcing steel stresses by 9% and 11%, respectively. The results of the parametric study also indicated that increasing the FRP plate cross-sectional area can reduce the maximum girder deflections and reinforcing steel stresses

by approximately 20% and 22%, respectively. Moreover, increasing the FRP plate modulus of elasticity was shown to reduce both the maximum girder deflections and reinforcing steel stresses by 16%.

Fiber-reinforced polymer wrap has been researched considerably with regards to the repair and strengthening of corrosion-damaged columns. Bridge columns are especially vulnerable to corrosion-induced deterioration due to their frequent exposure to deicing salts. In Wisconsin, concrete columns can be exposed to deicing salts through failed deck joints or from salt spray from passing automobiles or snowplows. It is a relatively simple process to clean and repair the damaged columns followed with encasement in FRP composite wrap. The wrap slows down the corrosion rate by preventing the ingress of deleterious substances and also by confining the concrete core, thereby providing it with strength and ductility.

In a variety of studies, this application of composite material has been proven to increase the service life of columns. Research conducted at the University of Toronto [29,39] studied the effectiveness of composite wrap to rehabilitate corrosion-damaged columns. The results of the study indicated that the composite wraps, being strong and corrosion resistant, proved to be effective as a column jacketing material. “The repair option that performed best, with regard to the post-repair corrosion rate, strength recovery, and deformation capacity, was also the simplest and easiest to implement alternative, consisted of cleaning the damaged surface (without removal of contaminated concrete) and wrapping layers of FRP sheets directly to the column surface [29].”

A field study conducted in Quebec involved the repair of the Highway 10 overpass columns [37]. The columns required repair due to corrosion-damage, primarily caused by the close

proximity of the highway lanes and the splashing of salt contaminated snow [37]. The project demonstrated that the relatively high costs of composite materials could be offset by a reduction in labor costs. The repair work required only three weeks time. The lighter weight of the material and the ease of application allowed a reduction in the number of workers as well as the number of labor hours. Other advantages observed were that formwork was not required for the column repair and the flow of traffic was not interrupted during the repair work.

FRP wraps have also demonstrated to be an effective alternative rehabilitation material for repairing and strengthening bridge piers. A bridge pier is exposed to a variety of loads, water currents, ice impacts, and corrosion attributed to deicing salts leaking through failed expansion joints or from the spray of salt-laden snow. Composites are often chosen to rehabilitate bridge piers due to their strength and durability.

A study conducted by Gergely et al [19, 20] involved the repair of a Highland Drive and Interstate 80 bridge pier in Salt Lake City. The thirty-five year old pier was severely corroded due to freeze-thaw action and the use of heavy deicing salts. The bridge had also experienced an increase in vehicle weight and traffic and lacked adequate seismic detailing. It was determined that the significant corrosion damage had reduced the initial capacity of the pier. Furthermore, it was concluded that the pier would experience severe damage in the event of a major earthquake. The rehabilitation of the bridge pier involved applying CFRP fabric on the columns, cap beam-column joint, and the cap beam haunches. Experiments were performed at the University of Utah and Utah State University to verify the repair design. It was found that the ductility of the column/pier was doubled and the shear strength of the wrapped joints

were significantly increased. The construction cost 20% less than conventional repair methods and only required one week to complete. It was concluded that, when compared to traditional repair techniques, the advantages of the composite wrap repair method include that it is fast, non-intrusive, and does not increase the weight of the pier.

Corrosion-damage of concrete bridge beam-ends commonly occurs in northern climates. Corrosion of beam-end reinforcement often occurs due to the failure of the overhead deck expansion joint and improper deck drainage. The resulting steel corrosion and the spalling of concrete in the bearing zone can cause irreversible damage to the beam-ends. Conventional rehabilitation of damaged beam-ends generally requires the complete removal of the damaged region, followed by reconstruction. Common issues with this repair procedure include reoccurring spalls due to inadequate bond between the new and existing concrete and the high cost and time required completing the repair. In addition, if drainage issues were not addressed, the repair would likely not be effective.

Fiber-reinforced composites can be applied, with relative ease, around a concrete beam end. However, to our knowledge research has not been conducted regarding the effectiveness of FRP wrap to rehabilitate corrosion-damaged beam-ends. Since concrete beam-end corrosion damage is frequently encountered in Wisconsin, the effectiveness of various traditional and state-of-the-art repair techniques for addressing this problem should be examined.

2.6 SUMMARY OF CORROSION REPAIR METHODS

Surface treatments, while reasonably effective over the short-term, have demonstrated limited effectiveness over the long term, unless they are applied prior to chloride contamination.

Cathodic protection, while effective, is not commonly employed due to the high component and maintenance costs as well as the complexity of the method. In addition, due to the possibility of hydrogen embrittlement, cathodic protection of prestressed concrete beams is not recommended. Research studies have established the effectiveness of FRP composites to prevent and mitigate corrosion-damage in concrete columns. However, to date, no research has been conducted in regards to their effectiveness to prevent and mitigate corrosion-damage in prestressed concrete bridge beam-ends. Therefore, since corrosion damaged beam-ends are frequently encountered in Wisconsin, the need for experimental work studying the effectiveness of various surface treatments, including fiber-reinforced polymer wraps, to protect and mitigate corrosion damage in beam-ends is essential.

2.7 VEHICULAR IMPACT DAMAGE

Corrosion damage occurs over a relatively extended period of time. However, damage to bridges can occur instantly by an applied force from an over-height vehicle or water born vessels. Vehicle damage can have serious consequences and include both damage to concrete and damage to the reinforcing or prestressing steel. A 1992 report by the Texas Department of Transportation [18] has suggested that with increasing demands on infrastructure and new bridges being built, the occurrence of over-height vehicle impact will continue to rise.

Depending on when the damage occurs, full-scale repairs may not be able to be performed immediately. In this case, temporary measures should be instituted to protect the bridge. These measures include the removal of all loose concrete and installation of a barrier beneath the damaged member to catch concrete that may be inadvertently dislodged. Also, weight

restrictions may be posted on the bridge to protect the most severely damaged members. The bridge should be monitored closely to prevent any further damage.

The amount of damage caused by vehicle impact can be classified as minor, moderate, or severe damage [18]. Minor damage consists of isolated cracks, nicks, shallow spalls, and scrapes. Moderate damage involves much larger cracks and spalls that expose undamaged reinforcing steel or prestressing tendons. Severe damage includes exposed, damaged steel and/or tendons and a significant loss of concrete cross section as well as girder distortion or lateral misalignment [18].

Repairs not only restore the structural integrity of the bridge, but also the appearance and durability of the damaged member. When the damage is classified as minor, the structural integrity of the bridge has not been compromised. The repairs are performed to restore the aesthetics and durability of the element. Typically, spot patching can fill cracks and spalls to protect the reinforcement from exposure [18].

Moderate damage is still considered non-structural. However, when reinforcing steel or prestressing tendons are exposed, all corrosion products should be removed and the steel should be treated with corrosion inhibitors before patching [18]. Splices of prestressing tendons, reinforcing steel and stirrups may be required if the members have lost significant amount of cross section due to corrosion. Any cracked, undamaged members should be epoxy injected to improve the durability of the element [18]. Cracks too fine to be injected should be treated with a silane sealer to prevent the ingress of deleterious substances [18]. It is recommended that the damaged element be loaded before the repair material is placed to ensure that recast concrete would regain prestress as originally intended.

Severe damage typically includes damage to the structural integrity of the member. A structural analysis of the bridge may be required to determine if the damaged member can be sufficiently repaired to return the bridge to its pre-damaged load-carrying capacity [18]. If prestressing tendons are severed, the tendons can be spliced by the following methods: external post-tensioning, internal splices, or metal sleeve splices. NCHRP Report 280 [43] provides a practice user manual for dealing with accidentally damaged prestressed concrete bridge members. The authors state in the reports that “they believe that sufficient research has been performed to document the effectiveness of the repair methods (listed above). No additional research is required prior to implementation of these methods in the field [43].”

External post-tensioning involves the use of high-strength rods or prestressing tendons jacketed against concrete corbels that have been recast against the girders. This method is suitable for splicing bundled strands or small tendons as well as a number of individual strands [43]. Figure 8 illustrates a standard external post-tensioning detail.

Internal splices incorporate a turnbuckle device to stress several strands. The device can be torqued to achieve the desired stress level in the strands. This method is inexpensive and easy to install [43]. Preloading must be employed in the spliced areas to restore compression in the concrete patch. Figure 9 illustrates a method for splicing a single ½ inch 270 K strand.

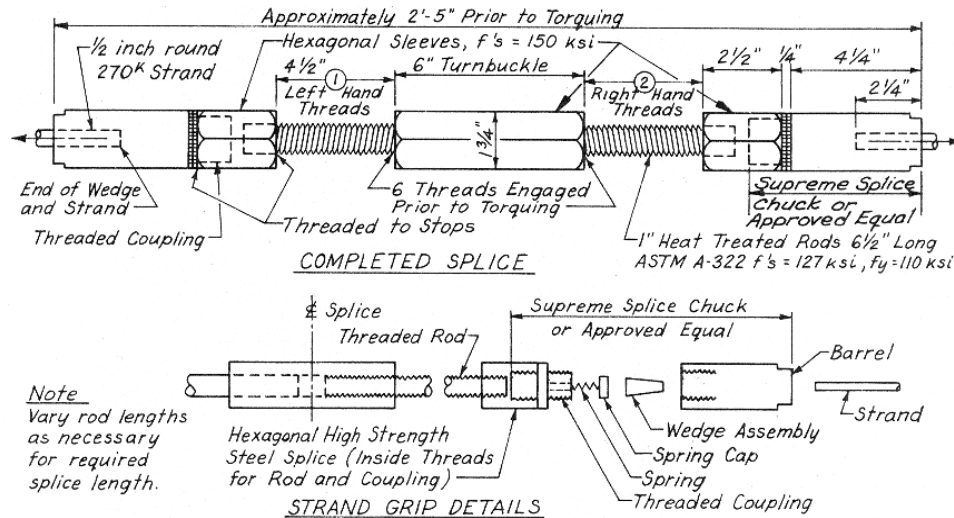


Figure 9. Single Strand Internal Splice [43]

Metal sleeve splices utilize metal plates bonded and/or bolted to the bottom and sides of a damaged girder. This method can be used to splice a large number of severed strands and when large volumes of loose or shattered concrete is present [43]. Figure 10 illustrates the use of a metal sleeve to splice ten severed 1/2 inch 270 K strands in an AASHTO Type IV beam.

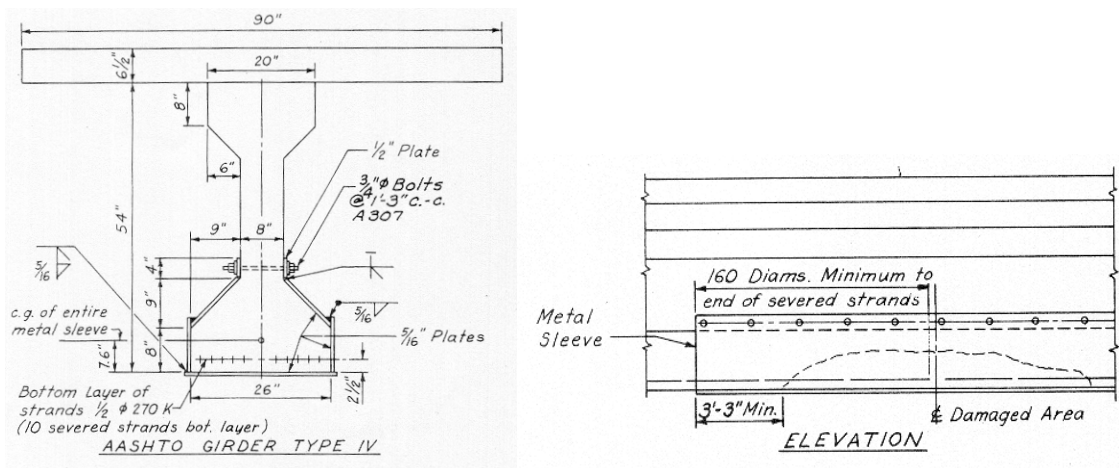


Figure 10. Metal Sleeve Splice [43]

Complete replacement of a member is normally the most expensive method of repair [43].

Replacing a member will require removing a portion of the roadway slab.

NCHRP Report 280 [43] recommends that the selection of a repair method should be based on an objective analysis. The selection of an appropriate repair method should be based on the type and extent of damage. Table 6 was developed to compare the difference between methods used to repair severely damaged girders.

Table 6. Severe Vehicle Impact Damage Repair Method to Consider [43]

Damage Assessment factor	Repair Methods to Consider			
	Post-tensioning	Internal Splicing	Metal Sleeve Splice	Replacement
Service & Ultimate Load	Excellent	Excellent	Excellent	Excellent
Overload	Excellent	Excellent	Excellent	Excellent
Fatigue	Excellent	Limited	Excellent	Excellent
Adding Strength to Non-Damaged Girders	Excellent	N/A	Excellent	N/A
Combining Splicing Methods	Excellent	Excellent	Excellent	N/A
Splicing Tendons of Bundled Strands	Limited	N/A	Excellent	Excellent
Number of Strands Spliced	Limited	Limited	Large	Unlimited
Preload Required	Perhaps	Yes	Possibly	No
Restore Loss of Concrete	Excellent	Excellent	Excellent	Excellent
Speed of Repair	Good	Excellent	Good	Poor
Durability	Excellent	Excellent	Excellent	Excellent
Cost	Low	Very Low	Low	High
Aesthetics	Fair*	Excellent	Excellent	Excellent

*Can be improved by extending corbels on fascia girder

N/A – not applicable

The durability of the repaired girders should be, as nearly as possible, equal to the durability of the original construction. NCHRP Report 280 [43] recommends that the following guidelines be considered in repairing damaged prestressed concrete bridge girders to achieve acceptable durability:

1. All unsound concrete should be removed and surface preparation should be such that new material placed will be compatible with existing material. New material should have equal or better strength characteristics than original.
2. Epoxy bonding, epoxy grout, and epoxy injection materials and systems should be fully tested and approved, and should be applied by trained personnel. Particular requirements concerning ambient temperatures must be observed.
3. Additional reinforcement to bond new material to existing surfaces should be considered.
4. Preloading should be used (if necessary) to ensure that the repair section would not be subject to greater tensile stress under live load than the original section.
5. Additional prestress force as required ensuring repaired stress levels are no greater than original design stress levels.
6. To further increase durability, the repaired areas should be sealed with proven water retardant.
7. Where repair design dictates, commitment should be made to perform periodic preventative maintenance.

The repair cost of minor damage, such as nicks, spalls, scrapes, cracks, and exposed strands is relatively low. The cost of materials is relatively low, and agency personnel can normally perform the repair. The repair cost of minor and/or moderate damage (per girder) normally would not exceed 10% of the cost of replacing the girder [43].

The repair of severe damage, such as severed strands and major concrete loss, normally will require the services of a contractor. The cost of the repair depends on several factors, such as: the type of repair, traffic control measures, and the extent of damage. Because of these factors, there is not precise cost data available. It is estimated that the repair cost of severe damage (per girder) will vary from 15% to 50% of the cost of replacing the girder, depending primarily on the extent of damage [43].

Neale and Labossiere [37] described the application of composite materials for the rehabilitation of the Webster Parkade in Sherbrooke, Quebec. The composites were used to reinforce beams that did not conform to current standards concerning bend and/or shear capacities. Following the composite rehabilitation, the strength capacity increased 15% of the initial bending strength of the beams, and 20% of the original shear strength.

Nanni and Gold [35] studied the repair of impact damaged concrete beams with CFRP plates. An over-height vehicle damaged four prestressed girders of the bridge overpass on highway Appia near Terracina, Rome. The conventional steel reinforcement was clearly visible after the loose concrete was removed. The concrete section was restored with non-shrink mortar. After surface preparation, CFRP plates were adhered to the girders. Combined with preloading, the bonding of the external plates restored the prestress that was lost upon vehicle impact. Furthermore, the author states that the strengthening approach was easy to perform and resulted in significant improvement in the ultimate load capacity and, to a lesser extent, the flexural stiffness.

Some issues must be addressed before the application of FRP plates to repair impact damaged prestressed concrete girders can be implemented with widespread acceptance. The strength

and durability of the concrete-composite bond is critical to the success of the repair. It is necessary to avoid or at least limit the extent of FRP debonding in order to ensure the effectiveness of the strengthening repair and the ductility of the load-deflection response [50]. In addition, a high degree of quality control and quality assurance must be established during the installation of the repair. Other engineering issues that must be addressed are FRP materials low modulus of elasticity, low failure strain, and the fact that it cannot be bent after fabrication [47]. In addition, repairs employing FRP materials have a relatively high initial cost. FRP material can cost five times more than steel (by weight), but these numbers can be misleading since less FRP material can normally repair the same amount of concrete [47]. The extent of FRP composite applications will depend upon the resolution of these issues.

2.8 LITERATURE DATABASE

After completion of comprehensive literature review in the field of rehabilitation of concrete bridges, focusing especially on northern climates, was completed, an extensive literature database was developed using Microsoft Access. Over 570 papers and reports were cataloged, and include such searchable information as the title, publisher, author, and date. The database also includes the abstracts or summaries of many of the papers. The user can search the database by performing a keyword, title, or author query.

EXPERT SYSTEM SOFTWARE

3.1 BACKGROUND

Expert systems have shown to be a useful tool to aid in the decision making process for a variety of applications in the construction industry. These systems have been applied in the fields of structural design, distress diagnosis, or repair schemes identification. However, according to Kaetzel and Clifton [24], the success rate in using expert system technology to develop practical applications in the construction industry is relatively low. They attribute this to user attitude, constraints in acquiring sufficient knowledge about a particular subject, and lack of easy-to-use development tools. Therefore if realistic expectations and sufficient knowledge base are in place, an expert system cannot replace the expert, but can assist in the decision-making processes. The complexity of bridge condition assessment and subsequent identification of repairs could be made more manageable by an expert system that could aid in the decision making process.

3.2 EXPERT SYSTEM TOOLS

Expert systems are also referred to as knowledge-based or decision support systems that emulate human expertise. They are normally designed to mimic the role of an expert. The user is prompted by a series of questions and statements, which will lead to a final conclusion or recommendation.

An example of an operational expert system in use today is Highway Concrete (HWYCON) Expert System. The program was designed to be used by inspectors and engineers, and is reportedly being used by some U.S. states, local governments, and city transportation departments [24]. It was developed to assist in the diagnosis, material selection, and general repair activities relating to concrete structures. To operate HWYCON the user answers questions about the structure and its environment. The program then provides the user with a hypothesis or recommendation. The knowledge base of the system includes digitized photographs, drawings, facts, rules of thumb, explanatory information, and tables. HWYCON has reportedly also been used to assist students at the University of Illinois in the diagnosis of distress in highway concrete structures, the selection construction and repair materials, and direction on the use of materials and procedures for repair. Kaetzel and Struble 1995 [25] report that HWYCON is useful for teaching the fundamental aspects of determining methods and materials for construction and rehabilitation of concrete highway structures.

Another construction related expert system, from Japan, is The Bridge Rating Expert System [24]. This system is designed to provide a serviceability rating for bridge structures in Japan. The system reportedly addresses the durability, load carrying capacity, and serviceability of bridges by incorporating knowledge from experts, probability theory, and a relational database component. The objective of the system is to rate the bridge condition in categories ranging from safe to dangerous. The Bridge Rating Expert System is reportedly in the developmental stage [24].

3.3 DEVELOPMENT OF EXPERT SYSTEM (ConBAR)

The HWYCON program is significant because it is one of the first comprehensive efforts to apply expert system tools to highway condition assessments. However, an examination of the HWYCON program indicates that a number of areas of weakness can be identified, such as the following:

1. HWYCON program modules cover a very wide range of topics including structures, pavements, construction, materials, etc. However, perhaps because of its very wide breadth, its depth is somewhat limited and only handles problems of a very general nature. For example, only two or three questions are typically asked by the system before a problem is identified for a bridge structure.
2. The HWYCON program does not typically evaluate the extent or severity of a bridge problem.
3. HWYCON program does not generally suggest corrective actions for bridge problems.

These issues prompted the researchers to propose development of a bridge diagnosis program that focuses on concrete bridges, identifies the extent of the problem, makes recommendations, and incorporates the compiled rehabilitation literature database. The infrastructure and a basic form of the Concrete Bridge Assessment and Rehabilitation (ConBAR) software are therefore developed in this project. The objective in the creation of the ConBAR expert system was to provide an electronic guide that would help diagnose the problem(s), determine the extent of damage, and identify repair, rehabilitation, or preventative maintenance options for concrete bridges. This expert system will use data inputted by the

user and a series of answers to questions prompted by the system. ConBAR provides a number of possible solutions along with their pros and cons, a suggestion, or a hypothesis. Recommendations for additional tests or sources of information are supplied to confirm or refute the hypothesis. The current state of development of ConBAR includes the complete infrastructure required as well as a limited number of expert rules, which must be expanded and enhanced in future developments of this program.

ConBAR expert system addresses cracking, surface defects (such as honeycombing and blistering), spalling, corrosion, vehicle impact damage, alkali-silica reactivity (ASR), and chemical exposure. The system also considers exposure conditions, previous repairs, bridge age, inspection information and other factors. The program knowledge base includes: (1) facts and rules of thumb, (2) visual information such as photographs and drawings, (3) indirect access to a rehabilitation literature database and (4) descriptive statements.

A session consists of a series of questions and supplemental information presented on a computer screen. A typical screen display consists of questions followed by a list of possible answers (see Figure 11). Pictures and detailed descriptions are also included for some questions to assist in the answering process. The user indicates the desired answer by clicking the button next to the answer with the mouse or choosing from a pull down list. An “enter” push button is provided to direct the program to the next step. When the questions have been completed, the system attempts to provide a solution or recommendation based on the responses to the questions. Three examples using the ConBAR program are presented in Appendix A.

The screenshot shows a software window titled "FORM1" with a tan background. It contains several sections:

- ASR Section:** A header "ASR" is followed by the question "Is ASR (Alkali-Silica Reactivity) occurring?". Below this are three radio buttons: "Yes" (selected), "No", and "Not Sure". A callout box labeled "Indicates user's choice" points to this section.
- Description Section:** A text box contains a description of ASR: "Three requirements must be met for expansive ASR to occur: (1) reactive forms of silica or silicate in the aggregate; (2) sufficient alkali (sodium and potassium) primarily from the cement; (3) sufficiently available moisture in the concrete. If any one of these requirements are not met, expansion due to ASR cannot occur." Below this, it states "In its simplest form, ASR can be visualized as a two-step process:" followed by two chemical-like reactions: "Alkali + Silica --> Gel Reaction Product" and "Gel Reaction Product + Moisture --> Expansion". A final sentence says "Actual expansion occurs in the second step when the ASR gel reaction product swells as it absorbs moisture. Potentially expansive gel reaction product does not form unless the first step". A callout box labeled "Description of deterioration process" points to this text.
- Data Entry Section:** A button labeled "ENTER DATA" is shown. Below it is a section titled "Data Entry" with the question "Is the data entered correctly?". It has two radio buttons: "Yes - accept data" and "No - retry and correct". A callout box labeled "Indicates user has finished inputting response" points to the "ENTER DATA" button. Another callout box labeled "Allows user to change response" points to the "No - retry and correct" option.
- Navigation:** A button labeled "CONTINUE" is located at the bottom right.

Figure 11. Sample of Expert System Screen

ConBAR expert system was programmed using both Microsoft Visual Basic 6.0 and CLIPS 6.20 (C Language Integrated Production System) programming languages. CLIPS is an expert system tool developed by the Software Technology Branch, NASA/Lyndon B. Johnson Space Center. CLIPS is designed to facilitate the development of software to model human knowledge or expertise in a great variety of applications. It is a tool for the construction of rule and/or object based expert systems. CLIPS provides a "facts list" that includes known

information, a “knowledge base” that includes all the expert rules, and an “inference engine” that controls the execution of the rules (decides which rules are executed and when). Although CLIPS is a very powerful program, it is difficult for people who are unfamiliar with expert systems to run it. Therefore, a Visual Basic code was incorporated to more easily interface with the user to obtain the required information, and thus eliminate the need for the user to learn CLIPS. CLIP transforms the information collected into “fact lists” that are understandable by CLIPS. CLIPS then uses these “facts” to execute the expert rules previously written by the programmer. A CLIPS rule is similar to an IF/THEN statement in a procedural language like C or Pascal. Therefore if certain conditions are true then some rules “fire” and the selected actions are executed. CLIPS then returns the solution to Visual Basic where it is presented to the user and displayed.

EXPERIMENTAL PROGRAM

4.1 INTRODUCTION

Based on the results of the literature review, a test plan and repair concept were formulated to study the prevention and repair of corrosion damage to prestressed concrete beam ends due to chloride-laden water infiltrating through faulty bridge expansion joints. The objectives of the experimental program were to (1) determine the effectiveness of a sealer, epoxy coating, polymer (resin) coating, and FRP wrap in protecting against corrosion damage in new members and (2) to establish the effectiveness of these treatments and patch repairs in reducing/preventing continued corrosion in members that were already contaminated with chlorides.

The work plan included performing laboratory tests on five new 8-foot long prestressed concrete bridge I-beams. The beam-ends were subjected to wet/dry cycles of salt laden water (6% NaCl solution) to accelerate the corrosion process. In addition to the salt-water exposure, the beam-ends were subjected to galvanostatic accelerated corrosion methods to assist in quicker corrosion initiation time and to draw the chlorides into the concrete faster. Two cathodes (short length prestressing strands) were embedded in each beam end to facilitate reverse cathodic protection, thereby making the entire reinforcement system anodic. Selected end regions were pretreated with a sealer, coating, polymer coating, or FRP composite wrap to assess their effectiveness in protecting the beam when subjected to an accelerated corrosive environment. Some beam-ends were left untreated. After a time period of over six months,

some of the previously untreated beam-ends were patch repaired or subjected to one of the prior stated surface treatments, and the accelerated corrosion process was continued for all specimens.

4.2 SPECIMENS

Pretensioned concrete beam specimens consisted of new 8-foot long AASHTO Type II sections as illustrated in Figure 12. The beams contained 18 - ½ inch diameter grade 270 low relaxation seven-wire prestressing strands. The magnitude of force on each strand prior to prestress transfer was 75% of the guaranteed ultimate tensile strength or approximately 30,980 lbs. All strands were straight and were cut flush with the end of the beams. The beams also contained stirrups and other conventional reinforcement as shown in Figure 13 (details provided in Appendix B). The conventional reinforcement was Grade 60 ASTM A614 steel with actual yield strength of 70 ksi and actual tensile strength of 109.2 ksi, per mill certification report. Two additional unstressed prestressing strands (2 feet long) were embedded in each beam end. These strands were electrically isolated from the main cage (strands and stirrups) to serve as cathode bars and facilitate the accelerated corrosion process. Please see Appendix B for the detailed shop drawings. Utilization of new beams in lieu of existing or in situ bridge beams had the following advantages: (1) enabled better control over the time schedule of the project, (2) all beams were subjected to the same controlled laboratory environment, (3) allowed uniform chloride exposure to all specimens, and (4) allowed incorporation of galvanostatic accelerated corrosion methods.

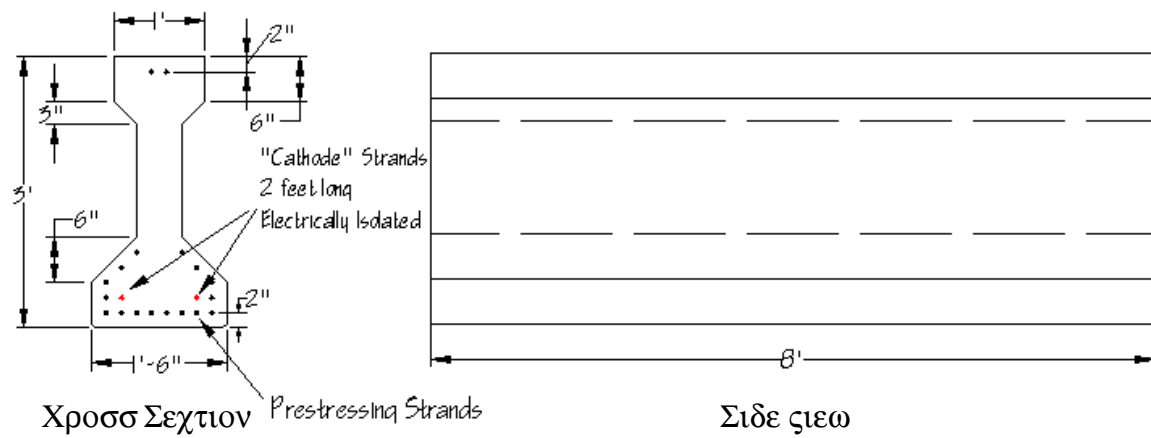


Figure 12. Design Details for Pretensioned Concrete Beam Specimens



Figure 13. I-Beam Steel Cage

All specimens were pretensioned and cast in January 2002 at Spancrete's production facility in Green Bay, WI. The steel was placed by the fabricators and verified by the investigators. Two cathodes were placed at each beam end. The investigators also verified the electrical continuity of the main steel cage and the electrical isolation of the cathodes prior to casting of concrete.

4.3 SPECIMEN EXPOSURE

After the beams were properly cured, they were delivered to the UW-Milwaukee Structures Laboratory. The indoor exposure regime was designed to simulate corrosion aging of prestressed concrete bridge beam-ends. The beams were positioned on neoprene pads on top of a constructed support system. Steel tube sections, with castors located at either end, supported the beams and a steel trough covered by roofing membrane (Figure 14). The support beams were built with castors to allow easy movement of beams in and out of their positions. The salt-water distribution system was constructed to subject the beams to controlled salt-water exposure. The system (illustrated in Figure 15) included the use of a water reservoir, located above the beams, which gravity fed the salt water to the beam-ends through a series of pipes, valves and hoses. The excess salt water was collected from each beam (trough system) and routed to a storage tank located in the building's basement. As needed, the water was then pumped back up to the reservoir. A photograph of the experimental set-up is shown in Figure 16. The beams were subjected to wet/dry cycles, which consisted of 4 days of exposure to salt water drip followed by 3 days dry. The salt-water exposure was designed to simulate the exposure commonly encountered in the field when the expansion joint fails and the bridge is subjected to deicing salt applications.



Figure 14. Beam Support System: front view (left) and side view (right)

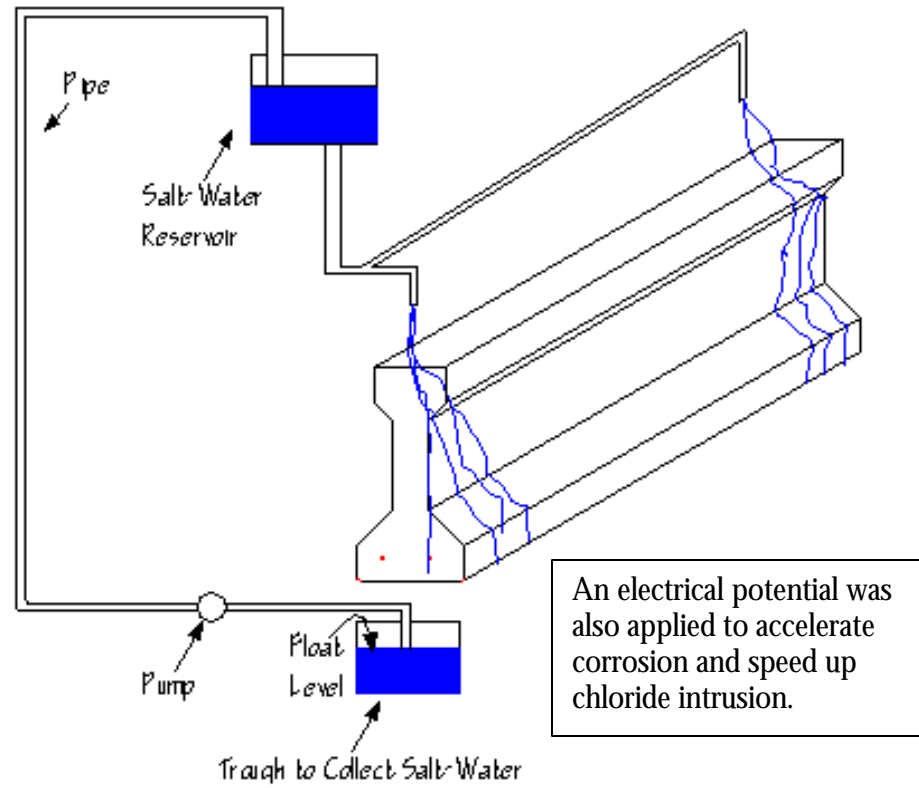


Figure 15. Specimen Exposure

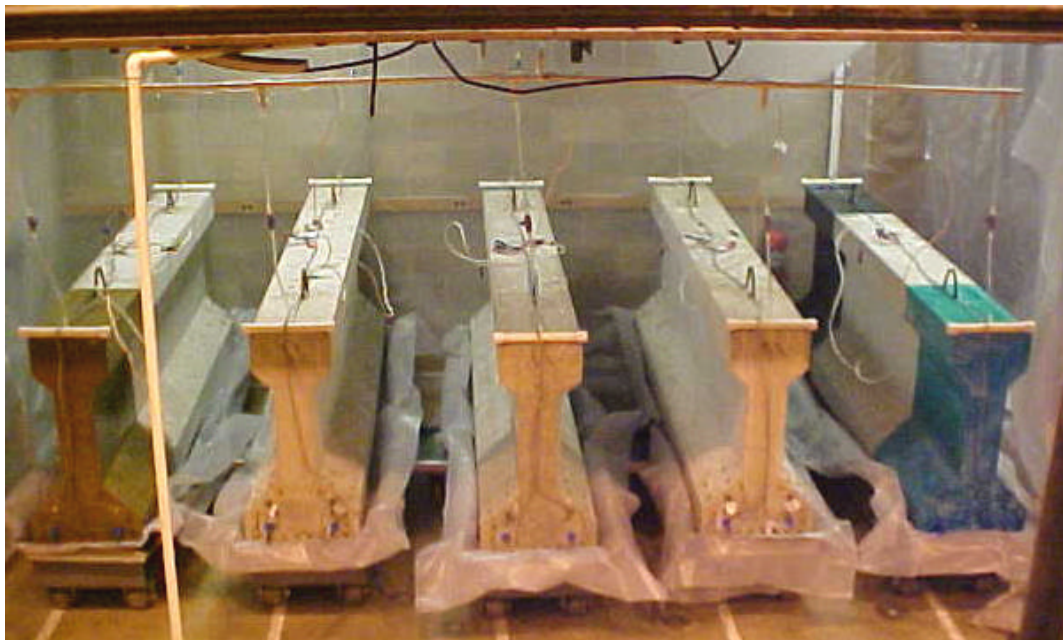


Figure 16. Initial Experimental Setup

After completion of the first accelerated corrosion cycle, the extent of corrosion damage of each beam was evaluated. Since the 6-month exposure did not result in the concrete spalling or significant tendon corrosion, the original exposure regime (Figure 16) was altered slightly to increase the likelihood of corrosion after the second cycle. Figure 17 illustrates the changes made to the system. Pipes (1 foot long) were added along the topsides of each beam end to allow salt water to flow along the side face of the beams. A larger pump was also added to facilitate the increased flow requirements.

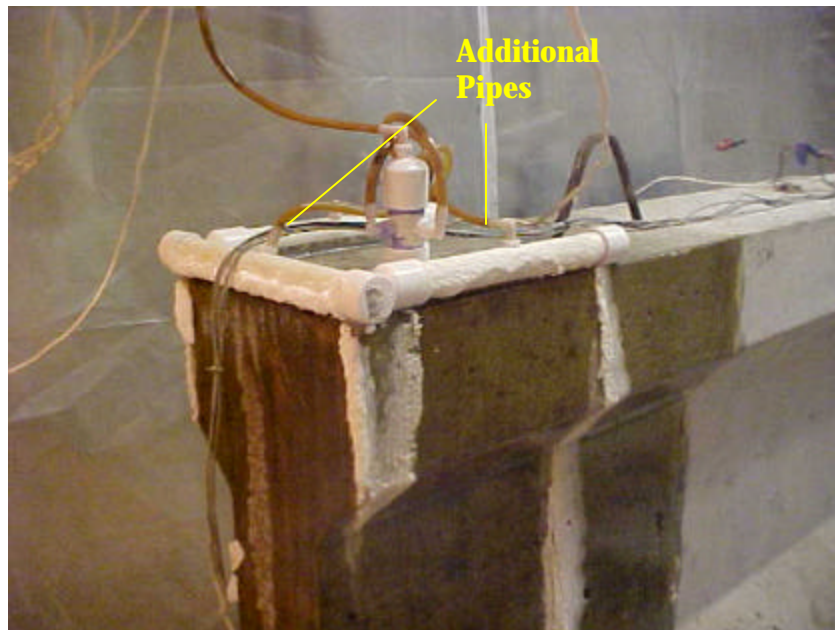


Figure 17. Final Experimental Setup

4.4 ACCELERATED CORROSION TESTING

Since the objective of the experimental program was to study the effectiveness of various protection systems to prevent or limit corrosion in prestressed concrete bridge beam-ends in a relatively short time period, it was necessary to rapidly induce corrosion in the specimens. All beams were subjected to the same accelerated corrosion regime. Accelerated corrosion was

achieved by subjecting the specimens to cyclic wetting and drying, involving a 6% sodium chloride solution, and applying a constant voltage to the steel cage. The specimen ends were exposed to 4 days of salt-water drip, followed by 3 days of no water exposure. Past studies have shown [52] that chlorides can be forced to migrate into concrete at a faster rate under the influence of an applied electric field. The impressed voltage applied to the steel cage attracts the negatively charged chloride ions towards the steel at a higher rate than the chlorides normally diffuse into the concrete. A regulated voltage of 9V was applied across the anode (steel cage) and the two internal cathodes. The schematic drawing of the accelerated corrosion cell is depicted in Figure 18.

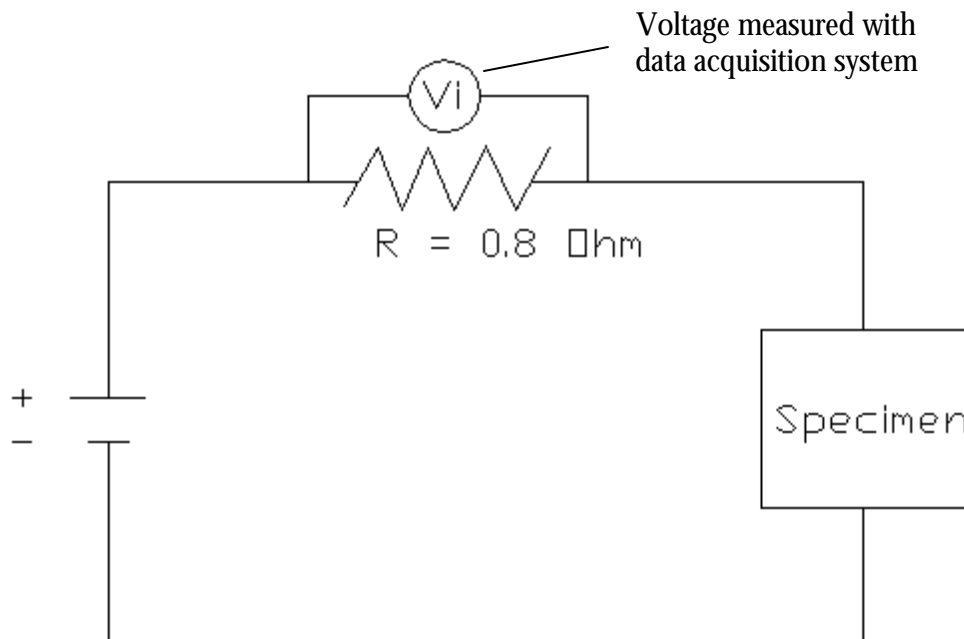


Figure 18. Corrosion Cell [23]

Lee [30] studied a similar accelerated corrosion regime at the University of Toronto. She subjected 12-inch diameter columns to an exposure regime that involved applying a 12V potential to the reinforcement cage of 12-inch diameter columns and subjecting them to 3% sodium chloride solution for wet/dry cycles of 1 day wet and 2 ½ days dry. In addition a

study conducted for FHWA [52] implemented a corrosion system exposing the specimens to 15% sodium chloride spray along with applying a current density of 700 mA/m². The investigators concluded that the past research supported the effectiveness of the accelerated corrosion regime described above. The total accelerated exposure period was approximately 18 months.

4.5 MONITORING

The corrosion current was monitored continuously throughout the duration of the accelerated corrosion regime with a DATAQ data acquisition system. Figure 19, next page, depicts the circuit used to apply an electric potential to the beams and connect the data acquisition systems to the specimens.

The total steel loss, w_t , (grams) during the given corrosion timeframe can be determined by integrating the curve of corrosion current versus time and using the following equation [30]:

$$w_t = \frac{At_m}{zF} \sum \Delta t I_{ave} \quad [\text{Eq. 4.5-1}]$$

where At_m is the atomic mass of the metal, z is its valency, F is Faraday's constant (96487 C/mol), Δt the time step, and I_{ave} is the average uniform current measured. For reinforcing steel, which is primarily iron, the atomic mass is 55.85 g/mol and the valency is 2.

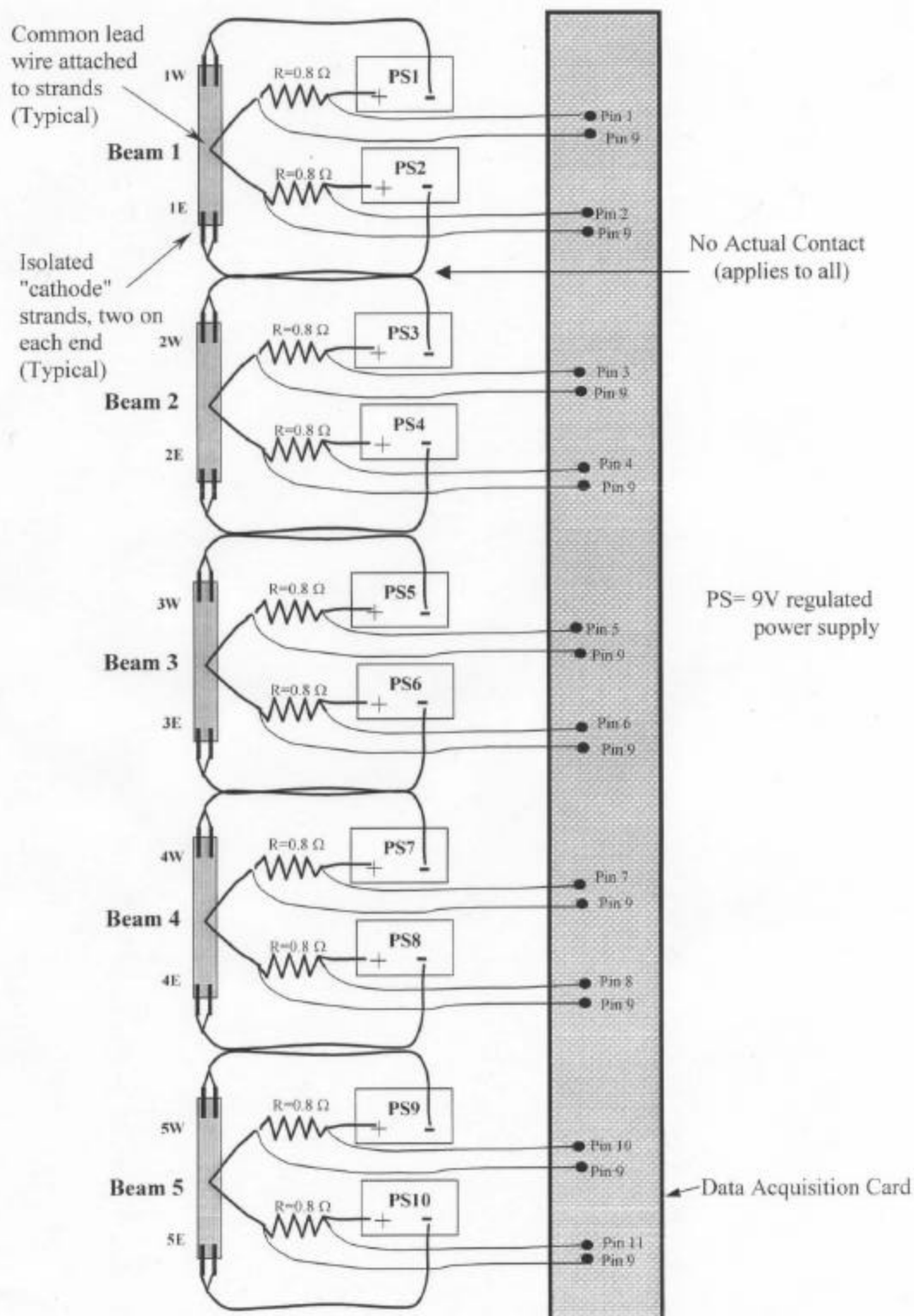


Figure 19. Wiring Diagram and Data Acquisition System

The chloride contents of the unexposed and exposed beams were determined by analyzing pulverized concrete samples at various depths. The initial chloride content measurements were taken at the center of a beam at $\frac{1}{4}$, $\frac{1}{2}$, $\frac{3}{4}$, and $\frac{7}{8}$ inch depths. The chloride contents after the first 6-month exposure cycle were measured on the bottom flange at 2 inches and 6 inches from the face of the beam at $\frac{1}{4}$, $\frac{1}{2}$, $\frac{3}{4}$, and 1 inch depths. At the end of the 1 $\frac{1}{2}$ -year exposure period, chloride contents (various depths up to 1 $\frac{1}{2}$ in.) were measured for all beam-ends on the sloping face of the bottom flange at a distance of 2 inches from the beam end.

Periodically, half-cell potential readings were taken. The potential difference between the surface of concrete and strands was detected by placing a copper-copper sulfate half-cell electrode on the concrete surface at different locations and measuring the potential difference between the steel cage and the concrete surface. The reference cell connected the concrete surface to a high-impedance voltmeter, which was also connected electrically to the steel cage. The voltmeter detected the potential difference at the test location. The half-cell reading would indicate the likelihood that corrosion was occurring. Half-cell readings were taken at twelve locations at each end of the beams and at one location in the center of each beam. The measurement points were spaced longitudinally at 6-inch increments and were located at center height of the surface being measured. The measurements were only taken on the non-treated beam-ends since surface treatments provide a non-conductive barrier that renders the half-cell measurements ineffective. A contour plot of the gathered data was developed for each region. Figure 20 illustrates the half-cell measurement locations.

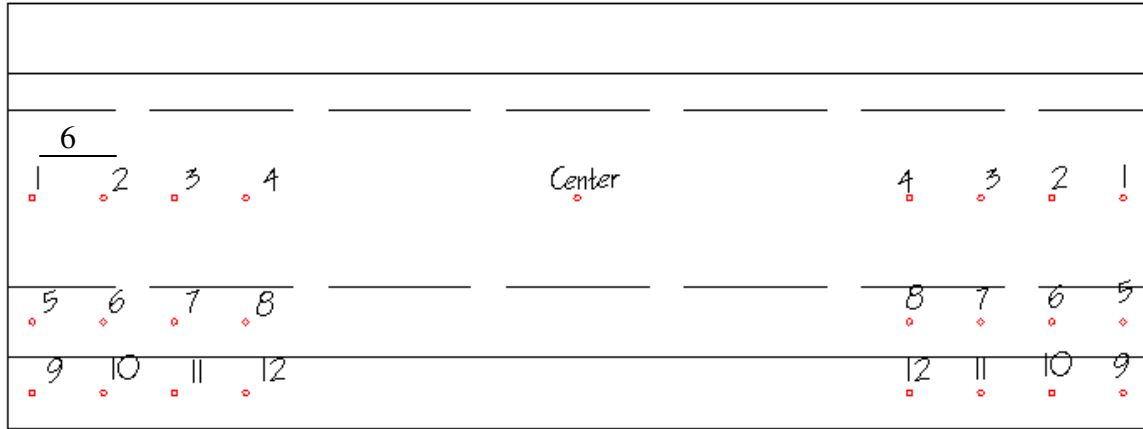


Figure 20. Half-Cell Measurement Point Locations

Expansion measurements were also periodically taken at each beam-end. Ten mechanical measurement points were attached to each side of the beam end at either 4-inch or 2-inch gage lengths. The measurement points were spaced longitudinally at 4-inch increments. A mechanical displacement-measuring device determined the expansion or contraction of the concrete to the nearest 10,000th of an inch. The expansion measurements were compared to readings taken from unexposed and untreated 4-inch and 6-inch cylinders, as well as a metal bar. Figure 21 illustrates the displacement measurement locations.

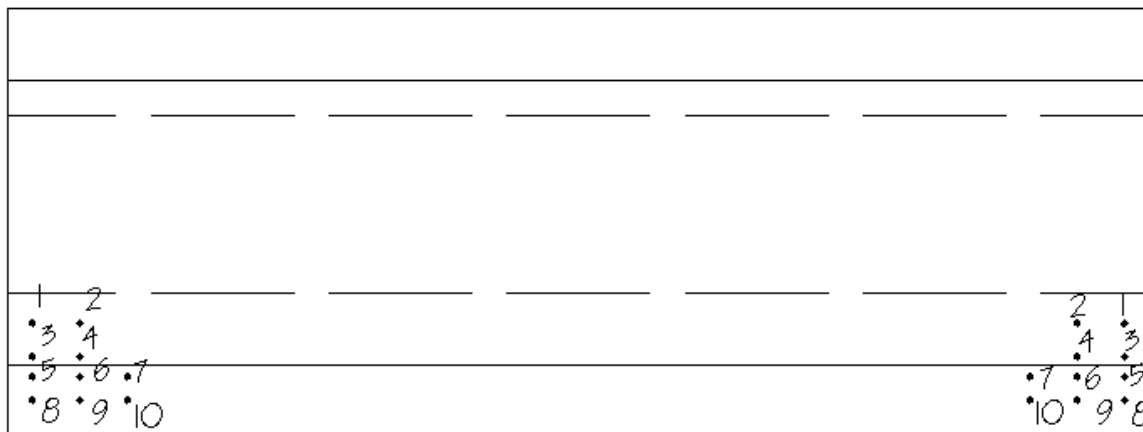


Figure 21. Displacement Measurement Locations

The specimens were visually monitored for cracking and spalling. Detailed crack maps were sketched at the end of each corrosion exposure cycle. The widths of the cracks were measured using a standard crack width comparator.

4.6 REPAIR MATERIALS USED IN THE EXPERIMENTAL PROGRAM

Selected specimens were designated for pretreatment with one of four surface treatments, while others were left untreated in order to be repaired and/or treated after completion of the first exposure cycle (detailed in test plan discussion). In addition, all concrete beam surfaces were prepared in the same manner prior to the application of treatments, which is detailed in section 4.8.

4.6.1 Carbon Fiber Reinforced Polymer (CFRP)

CFRP sheets were applied to one beam-end prior to the accelerated corrosion regime and to a second beam end after the first exposure cycle of the testing. The system employed was REPLARK 30 manufactured by Mitsubishi Chemical Corporation. It consists of the carbon fiber fabric, primer, putty, and resin. Since the system is lightweight and flexible prior to curing, the sheets can be installed around circular and square surfaces, as well as around irregularly shaped surfaces. In these tests, two fabric/resin layers were installed on the beams, with fiber orientation in the two layers at 90° with respect to each other. Figure 22 illustrates the installation of the FRP system.



Figure 22. Installation of CFRP System

Table 7 summarizes the properties of the carbon fiber sheet reported by the manufacturer. These properties are based on tests performed on laminate samples and are calculated using the net area method. The carbon fibers in the sheets are arranged parallel to one another and are held together with a thin weave of transverse glass fibers. The glass fibers do not contribute to the structural properties of the composite, but maintain the alignment of the carbon fibers during handling and installation. The sheets are also pre-impregnated in the factory with a small amount of resin to restrain the fibers [34].

Table 7. Carbon Fiber Sheet Properties

Properties	REPLARK 30
Fiber Areal Weight (lb/ft ²)	0.061
Thickness (inches)	0.0066
Tensile Strength (psi)	555x10 ³
Tensile Modulus (psi)	33.4x10 ⁶
Standard Width (inches)	13
Standard Length (feet)	328

Table 8 summarizes the properties of the primer, putty, and resin reported by the manufacturer. The primer penetrates into the concrete surface to increase the surface strength of concrete and to improve adhesion between the concrete and the carbon fiber sheet [34]. Primer PS401 is used for warm season applications with temperatures ranging from 68-95°F. The putty is used after the application of the primer to fill small holes, voids, honeycombs, pinholes, and other small surface irregularities to ensure a smooth final surface. The saturating resin is used to impregnate the reinforcing fibers, fix them in place, and provide a shear path to effectively transfer load between fibers and between the concrete substrate and fibers [34]. L700S-LS resin is used for warm season applications with temperatures ranging from 59-95°F.

Table 8. Primer, Putty, And Resin Properties

Property	Primer (PS 401)	Putty	Resin (L700S-LS)
Tensile Strength (psi)			>4200
Flexural Strength (psi)			>5500
Tensile Shear Strength (psi)			>1400
Adhesive Strength (psi)	>200	>200	>200
Compressive Strength (psi)		>7000	

The CFRP composite system is hand applied using a wet lay-up process. Dry, unidirectional, precut sheets of carbon fiber are impregnated with a saturating resin. The saturating resin, putty and primer bond the carbon fiber sheets to the concrete substrate. The laminate is formed using one layer of resin undercoat, one layer of carbon fiber sheet, and one layer of resin overcoat. The material properties of the REPLARK composite system as reported by the manufacturer are listed in Table 9. Section 4.8 details the surface preparation and application procedures implemented in this experiment.

Table 9. REPLARK 30 Composite Properties

Property	REPLARK 30
Thickness (inches)	0.0317
Tensile Strength (psi)	115×10^3
Tensile Modulus (psi)	6.9×10^6
Minimum Ultimate Breaking Load (lb/in)	3721
Guaranteed Ultimate Breaking Load (lb/in)	3675
Elongation (%)	1.7

4.6.2 Polymer (Resin) Coating

In order to assess the effectiveness of using only the polymer coating (P in FRP) of the composite system, two coats of the resin component of the RELPLARK 30 system (no fiber) were applied to one beam-end prior to the accelerated corrosion regime and to another beam end after the first exposure cycle of the testing. The properties of the resin coating are listed in Table 8 in section 4.6.1. The primer and putty were applied in the same manner as if the complete CFRP system was to be applied. Following the application of putty and primer, the first coat of resin was applied with a paint roller. After the first coat was tack free (3 to 4 hours) a second coat of the resin was applied.

4.6.3 Epoxy Coating

The coating used in this study was MASTERSEAL GP Epoxy Sealer. It is commonly employed to seal concrete surfaces to prevent deterioration such as spalling, scaling, cracking, and leaching. Test conducted by the manufacturer have reportedly shown that the coating could prevent over 94% of the chlorides in salt-laden water from entering concrete [11]. Table 10 summarizes the performance data of the coating as reported by the manufacturer.

Table 10. Coating Performance Data

Property	MASTERSEAL GP Epoxy Sealer
Reduction of water absorption into concrete (Test Procedure, NCHRP study, 12-19A)	91% minimum
Reduction of chloride content in concrete exposure test (Test Procedure, NCHRP study, 12-19A)	94% minimum
Solids (By weight)	50% minimum
(By volume)	58% minimum
Viscosity (mixed)	15 to 40 cps

MASTERSEAL GP could be applied with a squeegee, roller, or spray equipment to a clean, dry surface. A second coat was applied after the first coat became tack free (3 to 4 hours). Section 4.8 details the surface preparation and application procedures implemented in this experiment.

4.6.4 Sealer

The sealer used in this study was MASTERSEAL SL 40 VOC, a solvent based VOC-compliant silane penetrating sealer. The product creates a water repellent concrete surface, but still permits the concrete to breath. In addition, since it penetrates into the substrate, it generally does not alter the appearance of the concrete. Lastly, the manufacturer states that the sealer helps reduce efflorescence, atmospheric staining, and protects against damage caused by chloride intrusion [12]. Table 11 summarizes the performance data of the sealer, as reported by the manufacturer.

Table 11. Sealer Performance Data

Property	MASTERSEAL SL 40 VOC
Resistance to chloride (AASHTO T259 and T260)	Less than 0.22 lbs/yd ³ (criteria of 1.5) at ½" level Less than 0.00 lbs/yd ³ (criteria of 0.75) at 1" level
Average depth of penetration	0.22 inches (depending on substrate)
Water weight gain Absorbed Chloride (NCHRP 244 Series II Cube Test - 200ft ² /gal)	86% reduction – exceeds criteria 92% reduction – exceeds criteria
Moisture vapor transmission rate (OHD-L-35)	102%

The sealer was applied using a roller and paintbrush. Two coats were applied from the base of the beam up to ensure uniform distribution of the sealer. Section 4.8 details the surface preparation and application procedures implemented in this experiment.

4.6.5 Patching

In addition to surface treatments, the effectiveness of a patch repairs was also studied. Patch repairs involve removing portions of concrete and replacing it with some type of cement-based patching material. This type of repair is commonly used when large spalled or deteriorated regions need to be removed and repaired. Since spalling had not taken place at the time of patching, an area of the bottom flange in one previously untreated beam was removed to represent a spalled region. Section 4.8 details the surface preparation and application procedures implemented in this experiment. The patch material used in this study was “Vericoat Supreme”, a one component, microsilica and latex modified, nonsag repair mortar produced by Euclid Chemical Company. This cement-based product is designed for trowel

applied vertical and overhead repairs. Table 12 summarizes the properties of Vericoat Supreme as reported by the manufacturer [17].

Table 12. Vericoat Supreme Mechanical Properties

Property (28 day)	Vericoat Supreme
Compressive Strength (psi)	6200
Bond Strength (psi)	2100
Direct Tensile Bond Strength (psi)	310
Flexural Strength (psi)	650
Linear Shrinkage	-0.04%
Sulfate Resistance	+0.005%
Chloride Permeability (coulombs)	900
Working Time	30 minutes
Set Times (@ 70° F)	
Initial Set (hours)	1
Final Set (hours)	2 ½

Before application of the patch material a bond agent was applied to both the concrete and exposed steel surfaces. The bonding agent used in this study was “CORR-BOND”; a three part bonding agent composed of specialty water based epoxy and selected cementitious components produced by the Euclid Chemical Company. According to the manufacturer, this product facilitates a stronger bond between the existing and new concrete and provides protection against steel reinforcement corrosion. Table 13 lists the technical information of the bonding agent as reported by the manufacturer [16].

Table 13. CORR-BOND Technical Information

Property	CORR-BOND
Application Thickness (mils)	20
Slant Shear Bond to Concrete (psi)	
Open Time*	
0 hours	2000
12 hours	1950
Direct Tensile Bond to Concrete (psi)	
Open Time*	
0 hours	400
12 hours	350
7-Day Bond Strength (psi) (to wire brushed steel)	650

*Open Time: Time from the application of the CORR-BOND on 14-day old, hardened concrete until placement of the fresh concrete topping over CORR-BOND.

4.7 TEST PLAN

The test plan, detailed in Table 14 and illustrated in Figure 23 and 24, was employed to determine the effectiveness of various treatments to prevent prestressing steel corrosion. Two repair schemes were evaluated in this study. The first involved repairs where no concrete was removed and only a surface treatment was applied. Some specimens were treated with an epoxy coating, sealer, polymer coating, or CFRP composite wrap. The second repair scheme involved repairs where portions of concrete were removed and replaced with a patch material. Figure 25 illustrates the time period and repair method for each beam. End “A” indicates the west end of the beams and end “B” indicates the east end of the beams as they sat in the UWM Structural Laboratory.

Table 14. Laboratory Test Plan

Beam-End Treatment	1A	1B	2A	2B	3A	3B	4A	4B	5A	5B
Prior to Exposure										
Pre-Coated (epoxy coating)										
Pre-Sealed (silane sealer)										
Pre-FRP Wrap										
Pre-Polymer Coating (resin)										
No initial Treatment										
After Exposure Cycle										
Coating (Epoxy coating)										
Sealer (silane)										
FRP Wrap										
Polymer Coating (resin)										
Patch Repair Only										
Do Nothing										

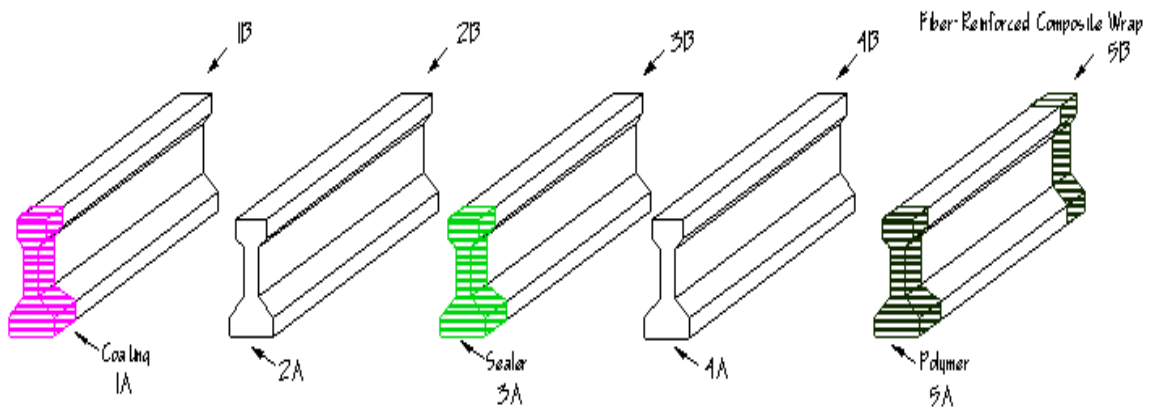


Figure 23. Laboratory Set-up Prior to Accelerated Corrosion

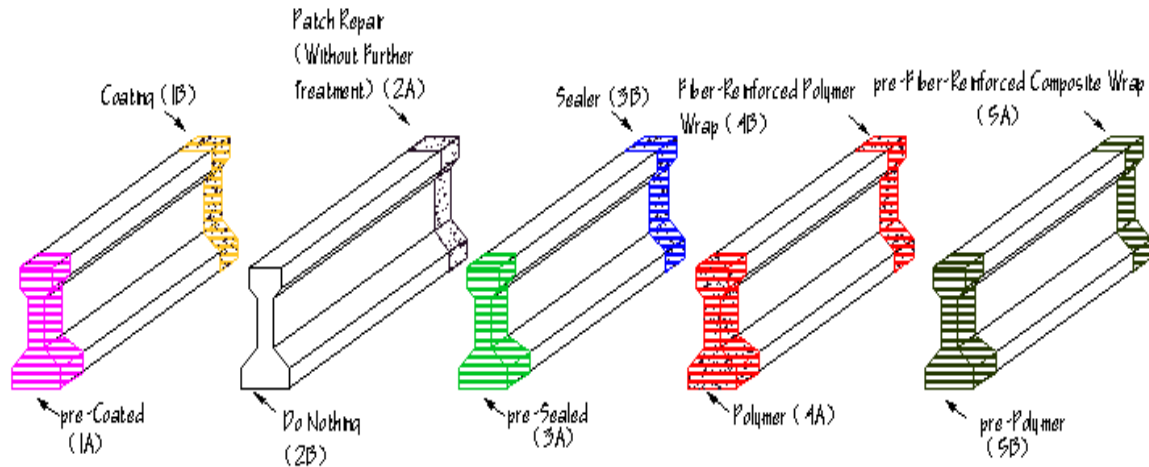


Figure 24. Laboratory Set-up After First Phase of Accelerated Corrosion

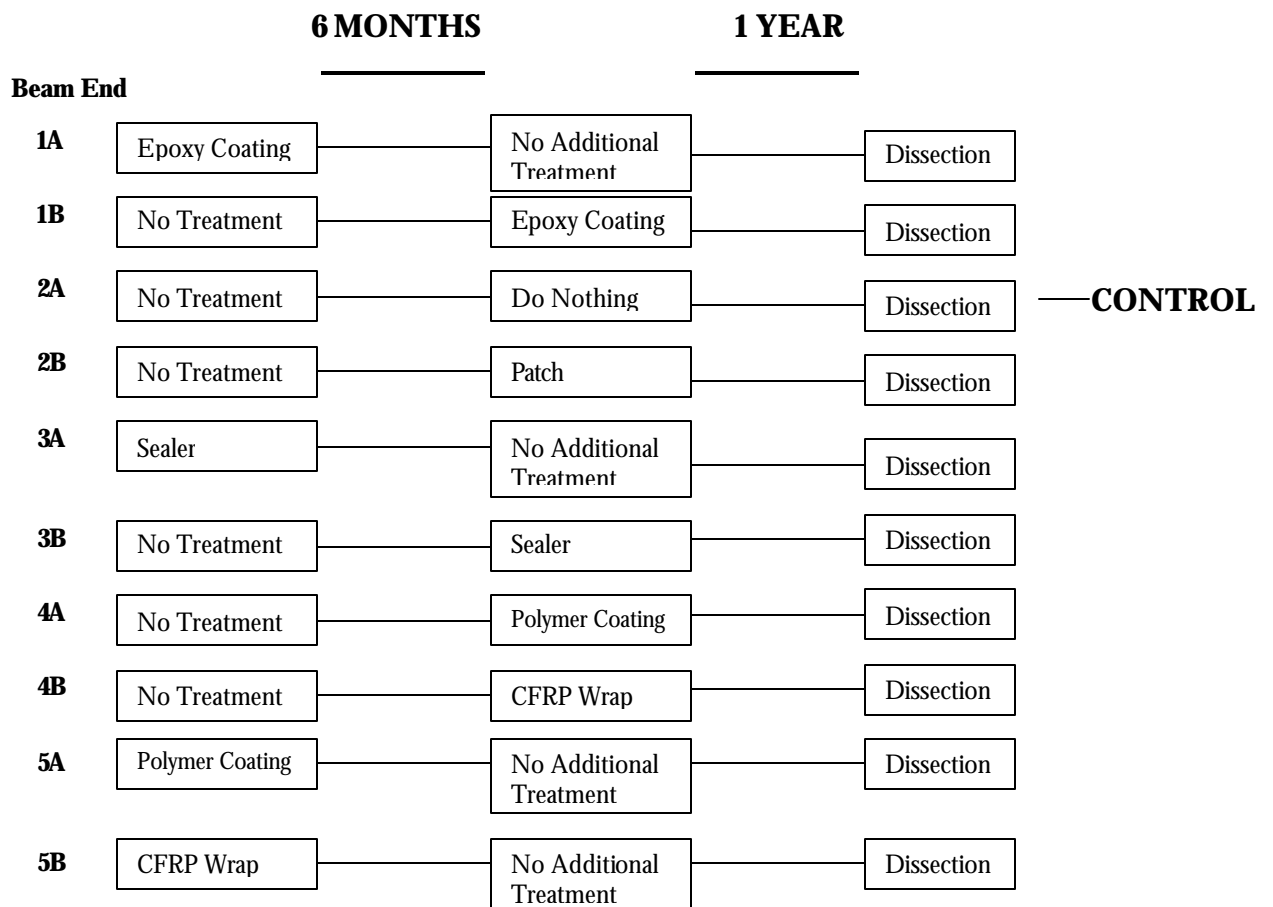


Figure 25. Repair Method & Time Period for Each Beam End

4.8 SURFACE PREPARATION & TREATMENT APPLICATIONS

Before exposure to the accelerated corrosive environment, four beam-ends (2-foot long sections in each beam end) were pre-treated with each type of surface treatment (i.e. silane sealer, epoxy coating, polymer resin coating, CFRP wrap). The surfaces were prepared by first grinding the surfaces of concrete, followed by thoroughly washing the surfaces to remove all accumulated dust and debris. After the surfaces were dry, an air hose was used to remove any remaining particles. The 2-foot long end sections for each beam end received their surface treatment. Manufacturer's instructions were followed in the application of the treatments. Table 15 summarizes the application rates and procedural notes of each material. The epoxy coating, resin coating, and silane sealer were applied with a paint roller.

Table 15. Surface Treatment Application Information

Surface Treatment	Notes
Epoxy Coating	Applied 2 coats
Silane Sealer	Applied 2 coats
Polymer Coating (resin)	Applied 2 coats after application of primer and putty
CFRP Wrap	Applied 2 layers (resin-sheet-resin-sheet-resin) after application of primer and putty

After exposure to the accelerated corrosive environment (over six months of exposure), the specimens subjected to surface treatments were allowed to completely dry. The same surface preparation and application procedure as stated previously was followed for the application of the various surface treatments.

Since the 6-month exposure did not result in spalling of concrete, it was determined that an 18-inch long concrete region was to be removed (Figure 26) for installation of the patch repair. A masonry saw was used to cut around the perimeter of the repair area to a depth of $\frac{1}{2}$ inch at a 90° angle to the surface. A series of cuts were made inside the repair region to allow for removal of the concrete with a chipping hammer. The chipping hammer was used to chip out the concrete in the repair area allowing for a $\frac{3}{4}$ inch clearance behind the first layer of strands exposed. After all the concrete was removed from the repair region, the concrete and steel surfaces were cleaned with a wire brush followed by cleaning with an air hose to remove any loose particles or debris. The surfaces of both the steel and concrete were covered with two coats of a bonding agent (CORR-BOND). The patch material was installed by a trowel. The region was then moist cured under wet burlap and covered by polyete for 3 days.

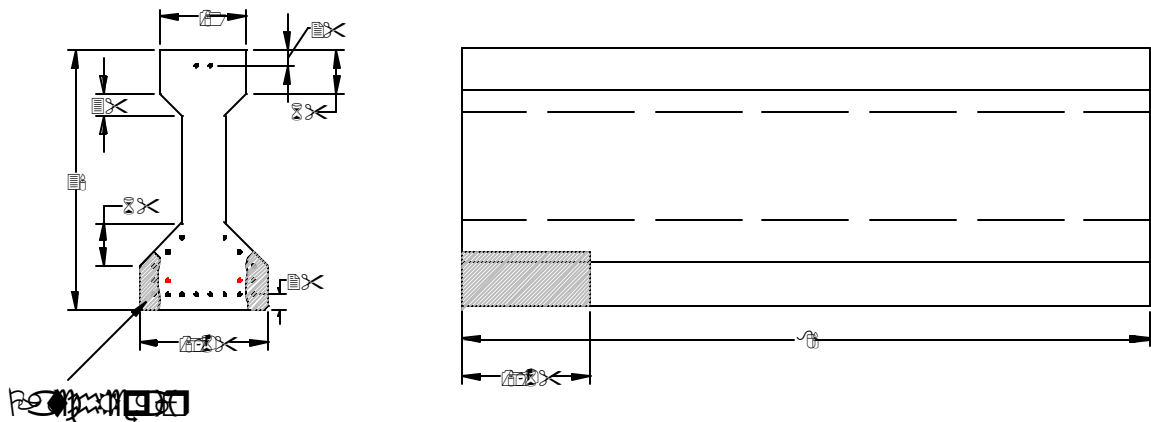


Figure 26. Beam Cross-Section with Patch Repair

All specimens were returned to the test area after the repairs were made and the surface treatments applied. The accelerated corrosion current and the salt-water exposure were re-initiated once the entire salt-water system had been cleaned and re-tested. The results of the monitoring program are presented in Chapter 5 of this report

EXPERIMENTAL RESULTS AND DISCUSSION

5.1 DATA GATHERED

Two repair schemes were evaluated in this study. The first involved repairs where no concrete was removed and only a surface treatment was applied. Some specimens were treated with an epoxy coating, silane sealer, polymer (resin) coating, or CFRP composite wrap. The second repair scheme involved repairs where portions of concrete were removed and replaced with a patch material (see section 4.6 for the test plan details).

The corrosion current was monitored continuously throughout the duration of the accelerated corrosion regime with a data acquisition system. In addition, periodically, half-cell potential readings were obtained. The half-cell measurements were only taken on the non-treated beam-ends since surface treatments provide a non-conductive barrier that renders the half-cell measurements ineffective. A contour plot of the gathered data was developed for each region.

Expansion measurements were also periodically taken at each beam-end. The expansion measurements were compared to readings taken from unexposed and untreated 4-inch and 6-inch cylinders, as well as a metal bar. However, due to problems encountered with the metal points corroding, the accuracy of the measuring device, and issues with keeping the points attached to the concrete surface, it was determined that the readings were inconsistent and not representative of accurate strain measurements.

The specimens were visually monitored for cracking and spalling. Detailed crack maps were sketched at the end of the first 6-month corrosion exposure cycle and at the end of the 1 ½-year exposure program. The widths of the cracks were measured using a crack width comparator. Chloride measurements were taken before exposure to chlorides, after 6 months of exposure, and at the conclusion of testing. The beam-ends were dissected and prestressing strands were exposed after a total of approximately 18 months of accelerated corrosion and exposure to chlorides.

5.1.1 Concrete Material Data

The measured slump of the concrete was 7 ½ inches. The average measured compressive strengths were 6598 psi at release of the tendons and 7530 psi after 28 days. Tables 16 and 17 summarize the average compressive strength results for the concrete cylinder samples.

Complete test results are listed in Appendix C.

Table 16. Concrete Cylinder Average Compressive Strength at Release*

Date	Age (days)	Cylinder Size (inches)	Sample Size	Mean (psi)	Standard Deviation (psi)
01/10/02	1	4x8	8	6317	1771
01/10/02	1	6x12	2	6598	N/A

Table 17. Concrete Cylinder Peak Compressive Strength*

Date	Age (days)	Cylinder Size (inches)	Sample Size	Mean (psi)	Standard Deviation (psi)
02/07/02	28	4x8	8	6522	1326
02/07/02	28	6x12	2	6012	N/A

* Test results provided by Spancrete, Inc. (manufacturer)

5.1.2 Chloride Content

The chloride content of the unexposed beams was determined by analyzing pulverized concrete samples at various depths using Rapid Chloride Test (RCT) 1029 method [21]. The RCT measures the acid soluble amount of chlorides as a percentage of concrete mass. A specified amount of chloride powder was extracted and mixed with a vial containing 10 mL of extraction liquid. A potential reading was taken with the RCT chloride electrode and then converted to chloride content in percent of concrete weight using the provided calibration chart. The same procedure was followed for determining the chloride content after the first cycle of saltwater exposure and at the conclusion of the 18-month test period. The initial chloride sample (before accelerated corrosion) was taken at the center of one beam. Table 18 summarizes the collected chloride content data before application of the accelerated corrosion regime. (Please see Appendix D for complete chloride data.) The average depth shown in Table 8 refers to concrete powder collected from a distance of $\pm 1/8$ inch of the average depth. For example, the chloride content at average depth of $\pm 1/2$ inch refers to powder collected from depths ranging between $3/8$ and $5/8$ inches. The reasonably uniform readings indicate that chlorides were present in the concrete at time of mixing.

Table 18. Initial Chloride Content of Prestressed Concrete Beam

Average Depth (inches)	Chloride Content (% by weight of concrete)
0.25	0.035
0.50	0.051
0.75	0.041
0.875	0.055

The measured chloride content of the concrete prior to corrosion testing was relatively high (Table 18). Therefore, a number of previously unplanned tests were performed to identify the source(s) of chlorides. The chloride content of the mix water was tested with the RCT method to determine if the water used in the concrete mix was the source of chlorides. The chloride content of the 300 mL sample of Green Bay water was found to be 0.0017%. A sample of water from Milwaukee was also tested for comparison purposes and was determined to have a chloride content of 0.0014%. Therefore, the chloride contents of both water samples were relatively equal and contained a negligible chloride concentration.

A sample of coarse and fine aggregates that were utilized in the construction of the beams were obtained and tested. The acid- and water-soluble chloride contents of the coarse aggregate samples measured were 0.041% and 0.035% by weight of aggregate, respectively. The measured acid-soluble chloride content of the sand was 0.039%. These results indicate that the aggregates were the likely source of the relatively high levels of chlorides measured in the new concrete.

Chloride samples following the first 6 months of the accelerated corrosion regime were taken at two locations on the bottom flange of the beam-end receiving the patch repair. The measurements were taken at various distances from the surface, 2 and 6 inches from the back end of the beam. Table 19 summarizes the collected chloride content data after application of the first exposure cycle.

Table 19. Chloride Content of Prestressed Concrete Beam After First Exposure Cycle

2 inches from End of Beam On Bottom Flange		6 inches from End of Beam On Bottom Flange	
Average Depth (inches)	Chloride Content (% by weight of concrete)	Average Depth (inches)	Chloride Content (% by weight of concrete)
0.25	0.96	0.25	0.21
0.50	0.74	0.50	0.29
0.75	0.47	0.75	0.188
1.00	0.29	1.00	0.135

The measurements indicate high chloride concentrations near the surface, with the values decreasing with increasing distance from the surface. This is consistent with the behavior of chloride ions migrating into the concrete. Figure 27 illustrates the comparison of the collected chloride content data.

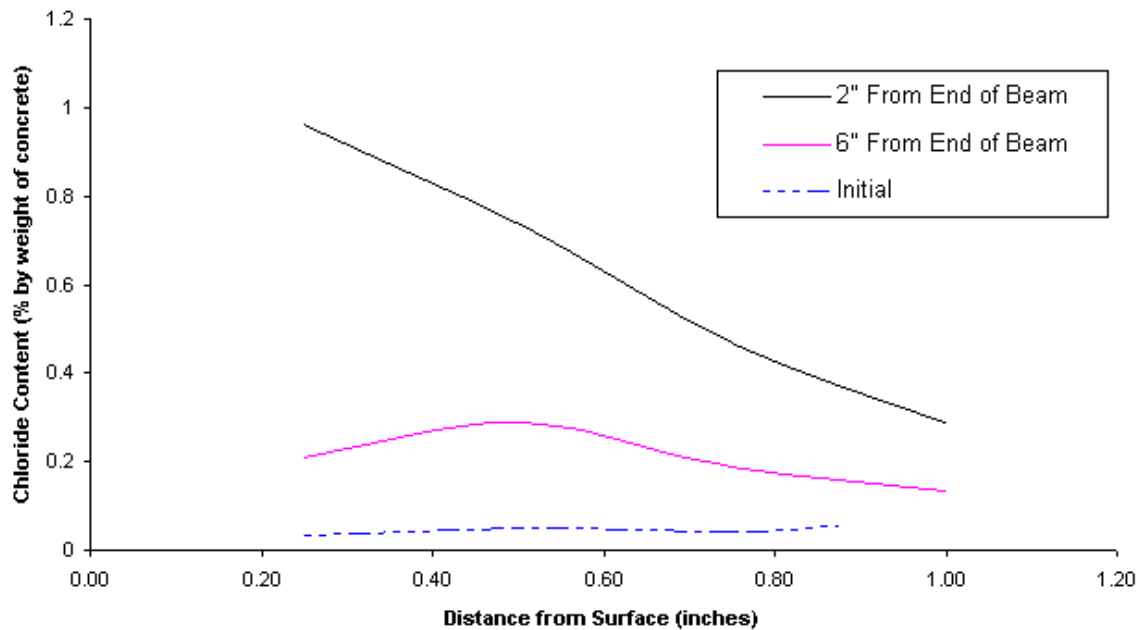


Figure 27. Comparison of Chloride Contents – Phase I

Research conducted by Lewis [26] suggests that the corrosion threshold value is 0.15% of acid soluble chloride by weight of cement. ACI Committee 222 [52] recommends the corrosion threshold value of 0.2% acid soluble chloride by mass of cement. Recently, other researchers have suggested a large variation in the corrosion threshold. In either case, the currently utilized corrosion threshold is exceeded at all depths measured at a distance of 2 inches from the end of the beam. At the location of 6 inches from the face of the beam, the corrosion threshold level is exceeded up to a depth of $\frac{1}{2}$ inch.

Chloride samples were also taken at all beam-ends at the conclusion of the entire 1 $\frac{1}{2}$ -year exposure prior to dissection. These measurements were made on samples taken in the middle of the sloping surface of the bottom flange at a distance of 2 inches from the back of the beam. Tables 20 through summarize the measured chloride contents on all beam-ends.

The highest chloride levels are observed in the beam-end with patch repairs. Acid-soluble chloride levels are on the order of 1.0% by weight of concrete is measured at depth of up to 1.0 inch. It appears that the interface between the old and new concretes may have allowed accelerated intrusion of chlorides deep into the patch and old concrete.

The chloride contents for the beam-ends that were treated with epoxy coating, polymer resin coating or FRP from day 1 clearly show significantly lower chloride contents than other specimens. The beam-end treated with Silane sealer from the first day had far less chlorides than the untreated beams (or beams treated after 6 months). However, the chloride levels for this beam-end were higher than the corresponding beams treated epoxy coating, polymer resin coating or FRP from Day 1.

The specimens that were treated after 6 months show high levels of chlorides, but they are less than the beam with patch repair. It should be noted that comparisons between measured chloride readings should be done in light of the fact that the chloride levels can vary statistically from point to point (even on the same beam at the same relative locations). Therefore, precision implied by the measurements at one location may be misleading, unless differences observed are significant.

The untreated beam-end 2A shows smaller chloride contents than expected. However, as noted in the corrosion current section of this report, a loose electrical connection may have somewhat reduced the corrosion potential to this beam end, thus explaining the lower –than-expected measured chloride content.

Table 20. Final Acid-Soluble Chloride Content of Prestressed Concrete Beam End 1A (Epoxy Coated from Day 1)

Average Depth (inches)	Chloride Content, 5 minute Test (% by weight of concrete)	Chloride Content, 24 Hour Test (% by weight of concrete)
0.25	0.071	0.072
0.50	0.058	0.058
0.75	0.057	0.058
1.00	0.080	0.081
1.25	0.070	0.072
1.50	0.075	0.078

Table 21. Final Acid-Soluble Chloride Content of Prestressed Concrete Beam End 1B (Epoxy Coated after 6 Months of Exposure)

Average Depth (inches)	Chloride Content, 5 minute Test (% by weight of concrete)	Chloride Content, 24 Hour Test (% by weight of concrete)
0.25	0.780	0.740
0.50	0.620	0.640
0.75	0.240	0.240
1.00	0.260	0.285
1.25	0.170	0.190
1.50	0.105	0.110

Table 22. Final Acid-Soluble Chloride Content of Prestressed Concrete Beam End 2A (No Treatment)

Average Depth (inches)	Chloride Content, 5 minute Test (% by weight of concrete)	Chloride Content, 24 Hour Test (% by weight of concrete)
0.25	0.830	0.840
0.50	0.460	0.465
0.75	0.205	0.215
1.00	0.105	0.110
1.25	0.140	0.145
1.50	0.100	0.100

Table 23. Final Acid-Soluble Chloride Content of Prestressed Concrete Beam End 2B (Patch Repair After 6 Months of Exposure)

Average Depth (inches)	Chloride Content, 5 minute Test (% by weight of concrete)	Chloride Content, 24 Hour Test (% by weight of concrete)
0.25	1.018*	1.018*
0.50	0.750*	0.750*
0.75	0.963*	0.981*
1.00	0.921*	0.965*
1.25	0.744*	0.710*
1.50	0.695*	0.709*

Table 24. Final Acid-Soluble Chloride Content of Prestressed Concrete Beam End 3A (Silane Sealer From Day 1)

Average Depth (inches)	Chloride Content, 5 minute Test (% by weight of concrete)	Chloride Content, 24 Hour Test (% by weight of concrete)
0.25	0.132	0.143
0.50	0.061	0.074
0.75	0.077	0.084
1.00	0.068	0.084
1.25	0.046	0.057
1.50	0.101*	0.103*

Table 25. Final Acid-Soluble Chloride Content of Prestressed Concrete Beam End 3B (Silane Sealer After 6 Months of Exposure)

Average Depth (inches)	Chloride Content, 5 minute Test (% by weight of concrete)	Chloride Content, 24 Hour Test (% by weight of concrete)
0.25	0.430	0.430
0.50	0.225	0.225
0.75	0.105	0.105
1.00	0.181*	0.184*
1.25	0.134*	0.136*
1.50	0.127	0.133

Table 26. Final Acid-Soluble Chloride Content of Prestressed Concrete Beam End 4A (polymer Resin Coating After 6 Months of Exposure)

Average Depth (inches)	Chloride Content, 5 minute Test (% by weight of concrete)	Chloride Content, 24 Hour Test (% by weight of concrete)
0.25	0.840	0.870
0.50	0.500	0.510
0.75	0.350	0.360
1.00	0.297*	0.318*
1.25	0.470	0.480
1.50	0.470*	0.477*

Table 27. Final Acid-Soluble Chloride Content of Prestressed Concrete Beam End 4B (FRP Wrap After 6 Months of Exposure)

Average Depth (inches)	Chloride Content, 5 minute Test (% by weight of concrete)	Chloride Content, 24 Hour Test (% by weight of concrete)
0.25	0.330*	0.318*
0.50	0.364*	0.364*
0.75	0.285	0.290
1.00	0.118	0.123
1.25	0.115	0.120
1.50	0.105	0.109

Table 28. Final Acid-Soluble Chloride Content of Prestressed Concrete Beam End 5A
(Polymer Resin Coating Since Day 1)

Average Depth (inches)	Chloride Content, 5 minute Test (% by weight of concrete)	Chloride Content, 24 Hour Test (% by weight of concrete)
0.25	0.069*	0.078*
0.50	0.076	0.076
0.75	0.060	0.060
1.00	0.053	0.061
1.25	0.072	0.073
1.50	0.076	0.076

Table 29. Final Acid-Soluble Chloride Content of Prestressed Concrete Beam End 5B (FRP
Wrap Since Day 1)

Average Depth (inches)	Chloride Content, 5 minute Test (% by weight of concrete)	Chloride Content, 24 Hour Test (% by weight of concrete)
0.25	0.064	0.068
0.50	0.054	0.058
0.75	0.056	0.062
1.00	0.062	0.070
1.25	0.064*	0.068*
1.50	0.118*	0.118*

Table 30 shows the above chloride data in a slightly different form. The 24-hr chloride test data shown above are summarized by focusing on 0.75 inch and 1.50 inch measurements. The chloride levels at each of the two levels are given a numerical rating of 1 to 8. If the chloride content is between zero and 0.1%, then a rating of 1 is given, etc. For example, a chloride content of .35 would be given a numerical rating of 4. Chloride contents higher than 0.7% are given a rating of 8. As will be seen in the following sections of this report, other comparative performance measures (for cracking and corrosion) are also based on a numerical measure from 1 to 8. It is clear that that among beam-ends that were pretreated from the first day, the polymer resin coating and the Silane sealer were the most effective. The FRP wrap was very close behind. Among the beam-ends that were treated after 6 months of exposure, the Silane sealer and the epoxy coatings had the least chloride contents. The highest chloride contents were observed in the patched beam-end 2B.

Table 30. Comparative Chloride Content Ratings* for All Beam-Ends Based on 24-hr Data at 0.75 and 1.5 in. Depths

Beam-End	Rating at 0.75 in.	Rating at 1.5 in.	Ave. Rating
1A	1	1	1
1B	3	2	2.5
2A	3	1	2
2B	8	8	8
3A	1	1	1
3B	2	2	2
4A	4	5	4.5
4B	3	2	2.5
5A	1	1	1
5B	1	2	1.5

Ratings based on numerical rating from 1 to 8 (1 best, 8 worst)

Shaded rows correspond to beam-ends that were treated after 6 months of exposure.

5.1.3 Corrosion Current

A regulated voltage of 9V was applied continuously over the course of the exposure cycles to facilitate an accelerated corrosion process and speeding the intrusion of chlorides. Plots of the corresponding corrosion current versus time (for the data collected in the first 10 months of exposure) are illustrated in Figures 28-32. The completions of the first exposure cycles are indicated on the plots. The prefixes (pre, post) denote whether the treatment was applied before the start of the accelerated corrosion regime, or if they were applied after experiencing 6 months of exposure. These figures show periodic increases (spikes) in the corrosion currents. These are associated with temporary stoppages of voltage applications. The short-term increase in currents after restoration of voltage is also observed in tests done by Lee [30].

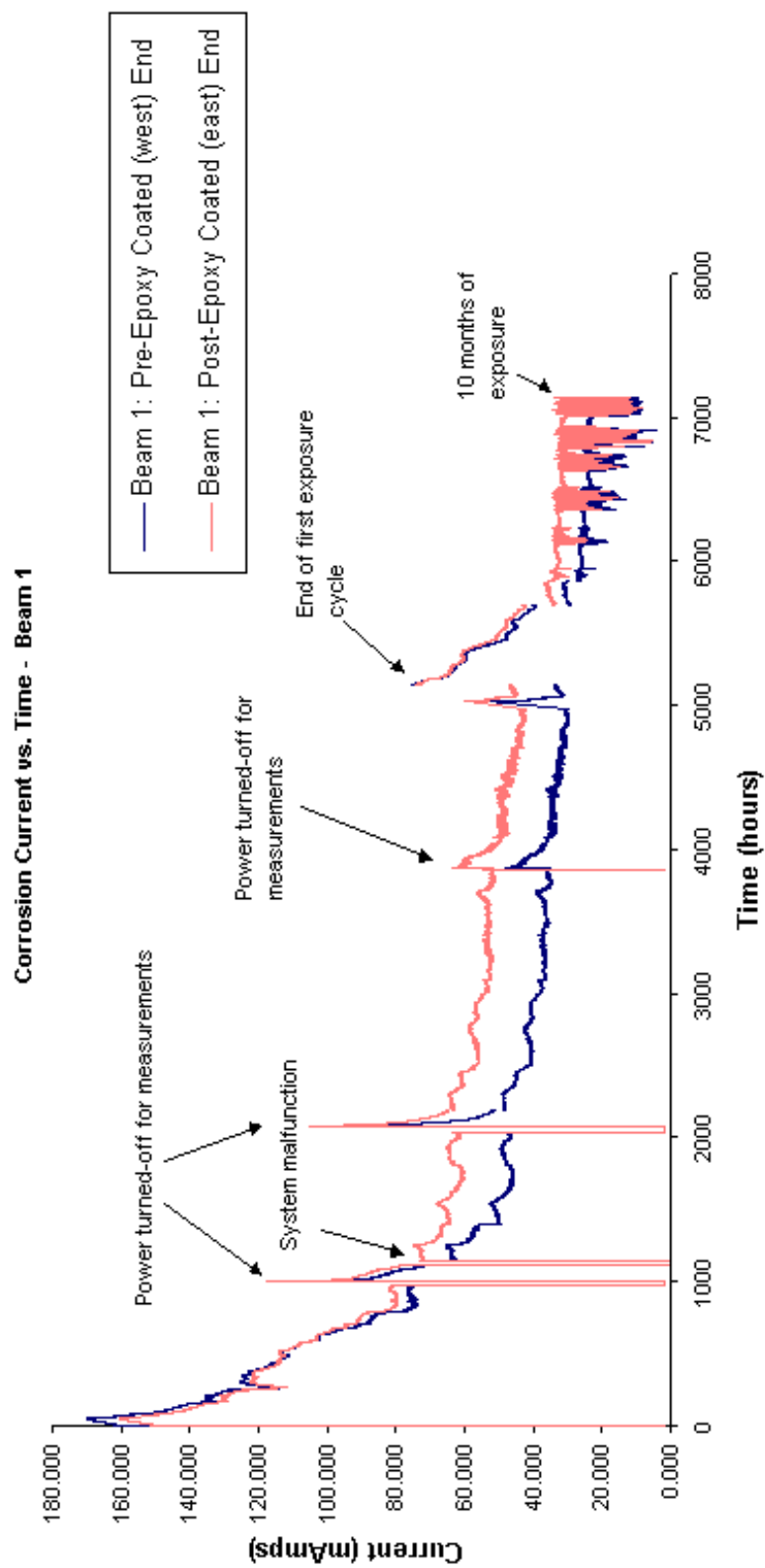


Figure 28. Corrosion Current vs. Time – Beam-Ends 1A (West) and 1B (East)

Corrosion Current vs. Time - Beam 2

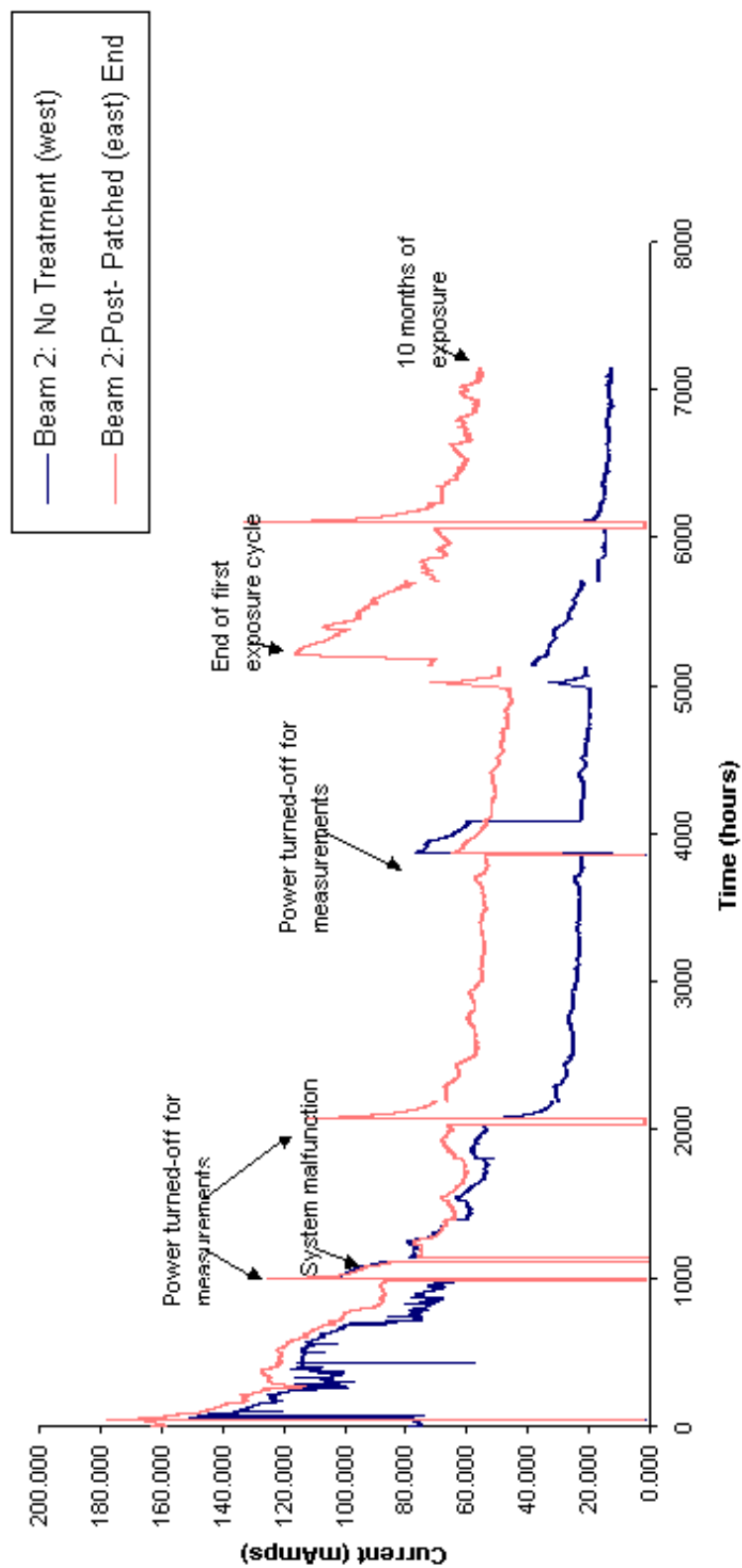


Figure 29. Corrosion Current vs. Time – Beam-Ends 2A (West) and 2B (East)

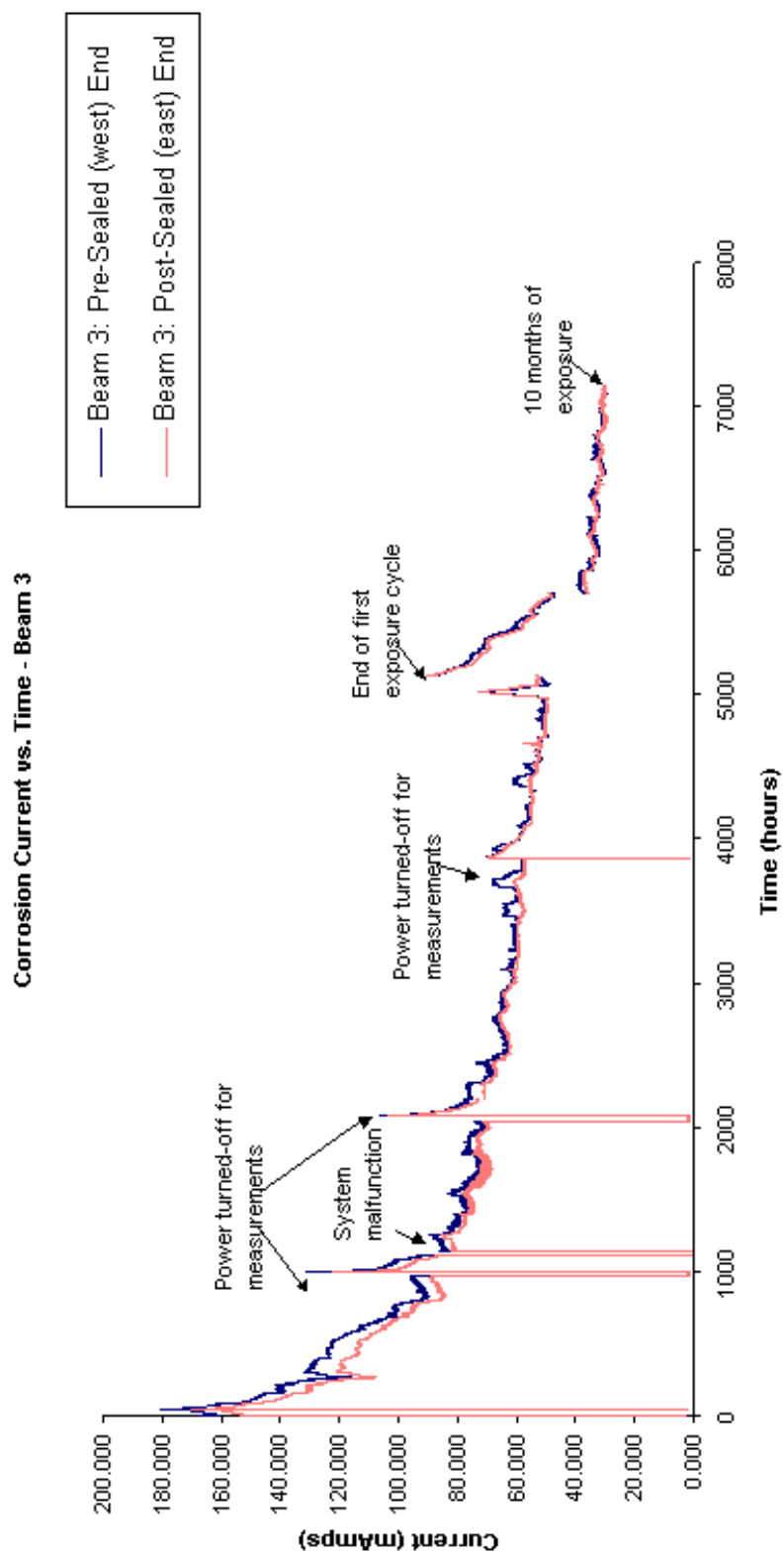


Figure 30. Corrosion Current vs. Time – Beam-Ends 3A (West) and 3B (East)

Corrosion Current vs. Time - Beam 4

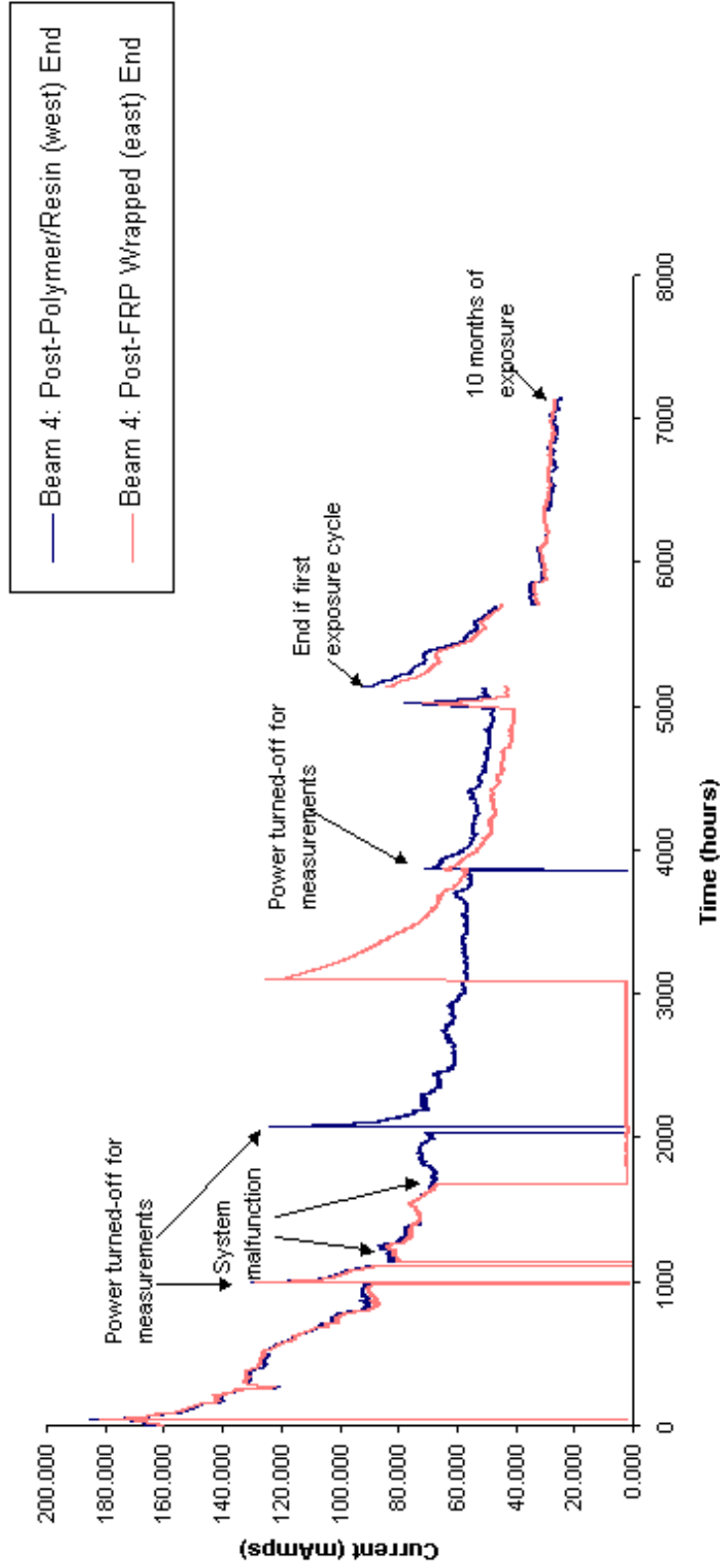


Figure 31. Corrosion Current vs. Time – Beam-Ends 4A (West) and 4B (East)

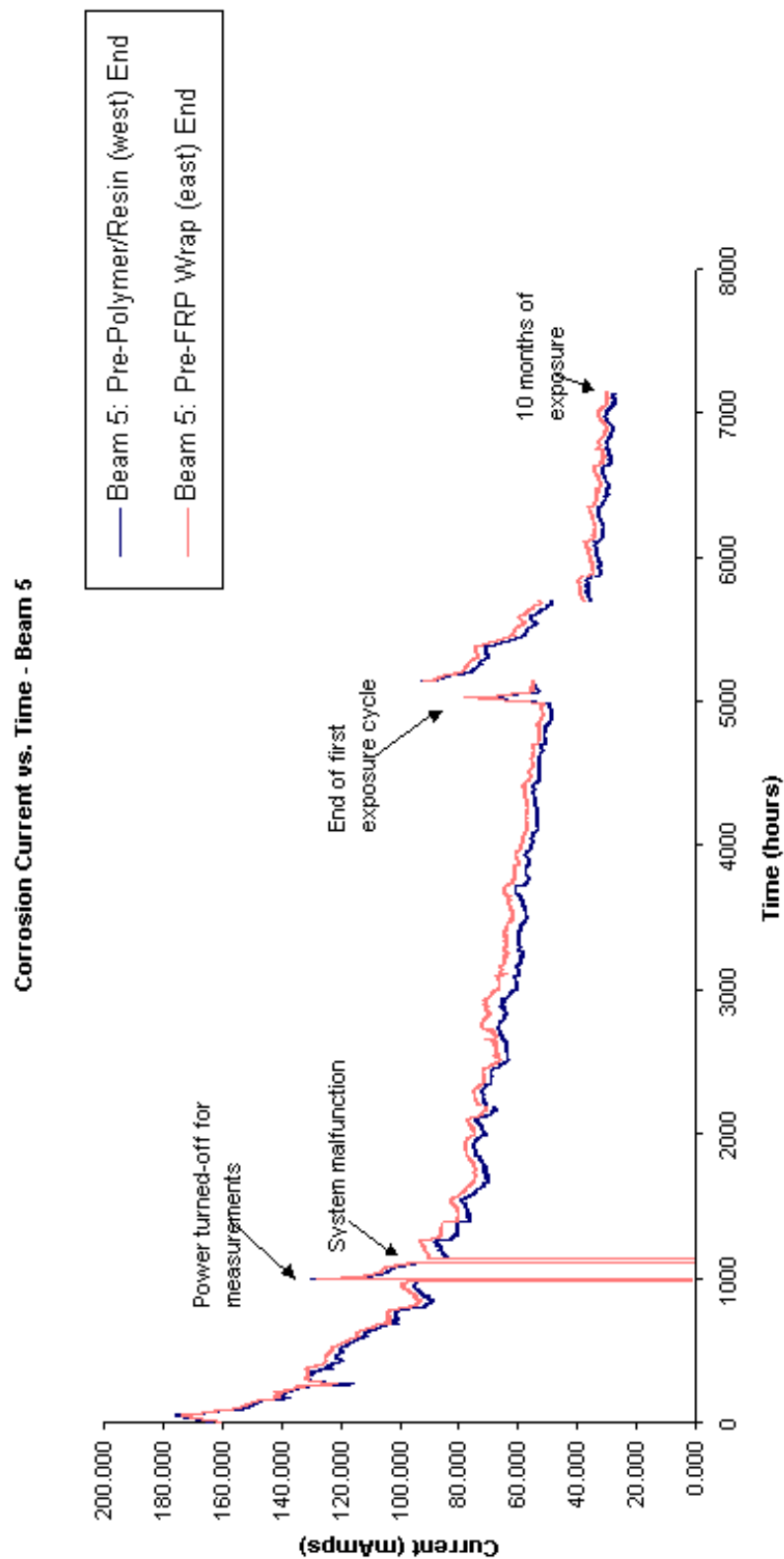


Figure 32. Corrosion Current vs. Time – Beam-Ends 5A (West) and 5B (East)

All curves exhibit a decrease in current over the 10 months of the experiment reported here. This reduction is commonly observed in such experiments, and is due in part to the fact that corrosion products increase the resistance at the surface of strands. Because of the exponential nature of the curve, the results obtained subsequent to the first 10 months of exposure are not plotted. The curves for the two ends of beam 1 are approximately similar until about the 800-hour mark, after which the curves begin to diverge. The pre-coated beam end (epoxy coated from Day 1) demonstrates a larger decrease in current at approximately 1500 hours in comparison to the untreated end. The cumulative area under the corrosion current versus time graph is indicative of the amount of steel loss due to corrosion. Since the pre-coated end has a smaller cumulative area under its curve, it can be deduced that this end is experiencing less steel loss over time. Therefore, the beam end treated with epoxy coating since Day 1 is experiencing less steel corrosion in comparison to the untreated (later coated) end.

The corrosion vs. time curves for the two ends of beam 2 are relatively similar until about 2000 hours into the experiment. After which, the “no-treatment” or west end experiences a significant decrease in current. Since these ends were exposed to the same exposure condition and both were initially untreated, their curves should be approximately equal for the first 6 months of the study. Given that the untreated end diverges so significantly, it can be deduced that there may have been electrical connection problems occurring with this end. It was observed that at the end of the first phase of exposure, there was only one undamaged connection between the beam end and the applied voltage. Therefore, the difference observed between the curves for the ends of beam 2 is more than likely due to electrical connection

problems, and does not reflect that less corrosion is occurring in the untreated, west end compared to the untreated, east end.

The corrosion current versus time curves for the two ends of beam 3 are generally similar for both the pre-sealed (west) end and the untreated, later sealed (east) end. This behavior is observed at the end of the first phase of the experiment and continues after 10 months of exposure. Since these curves are exhibiting similar behavior, it can be concluded that the pre-sealed end behaved the same as the initially untreated end. Therefore, this data seems to indicate that the penetrating sealer did not have a noticeable effect on preventing corrosion and the subsequent steel loss in the beam.

The corrosion current versus time curves for the two ends of beam 4 are nearly the same for both initially untreated ends. The exception is between approximately 1700 and 3000 hours, where the current drops to zero for the post-FRP wrapped (east) end. The drop in current was due to a loose connection between that end and the applied voltage. When the drop in current was observed, the connection was evaluated and remedied. After the readings stabilized once the connection was reestablished, the behavior of both curves returned to be nearly equal. Since both ends were initially untreated and subjected to the same exposure conditions, the graphs should display nearly the same behavior. For the 10 months of total collected data, the curves continue to be approximately equal after the polymer (resin) and FRP wrap was applied.

The corrosion current versus time curves for the two ends of beam 5 are nearly the same for the 10 months of collected data. The end pretreated with the polymer (resin) has a slightly less corrosion current in comparison to the end pretreated with FRP wrap (east). Form the

existing data collected, it can be concluded that both ends are experiencing similar corrosion damage, and therefore have similar effectiveness.

5.1.4 Effect of Time on Corrosion Rates in Field Structures

Vu and Stewart [49] developed a relationship between time since corrosion initiation and corrosion rate. The author states that corrosion rates predicted by his model appear to be reasonable and within the range of typical corrosion rates found in literature and therefore the model error is not expected to be high. However, the model is subjected to limitations since it has been validated with minimal experimental data, and the data that was collected was over a short period of time. A graphical representation of their model is illustrated in Figure 33. The vertical axis refers to the ratio of corrosion current at any particular time to the initial corrosion current. This representation is generally similar in shape to the experimental data observed.

The time it takes for the corrosion current to be reduced by 50% is approximately 8 years, or 70,080 hours. On average, the corrosion current data collected from this experiment shows the time it takes for the corrosion current to be reduced 50% is approximately 1250 hours, or 0.14 years. In other words, the accelerated corrosion regime compressed the time to initiate corrosion. If it is assumed that the relationship is similar in both cases (which may not be a sound assumption), then an estimate of the amount of time compression can be made. In this case, it could be estimated that 10 months of exposure in the laboratory has simulated 40 years in the field. However, the measured response after power shutdowns indicates that the rate of reduction can be artificially high in some cases. Therefore, the time compression ratio cannot be conclusively established based on available data.

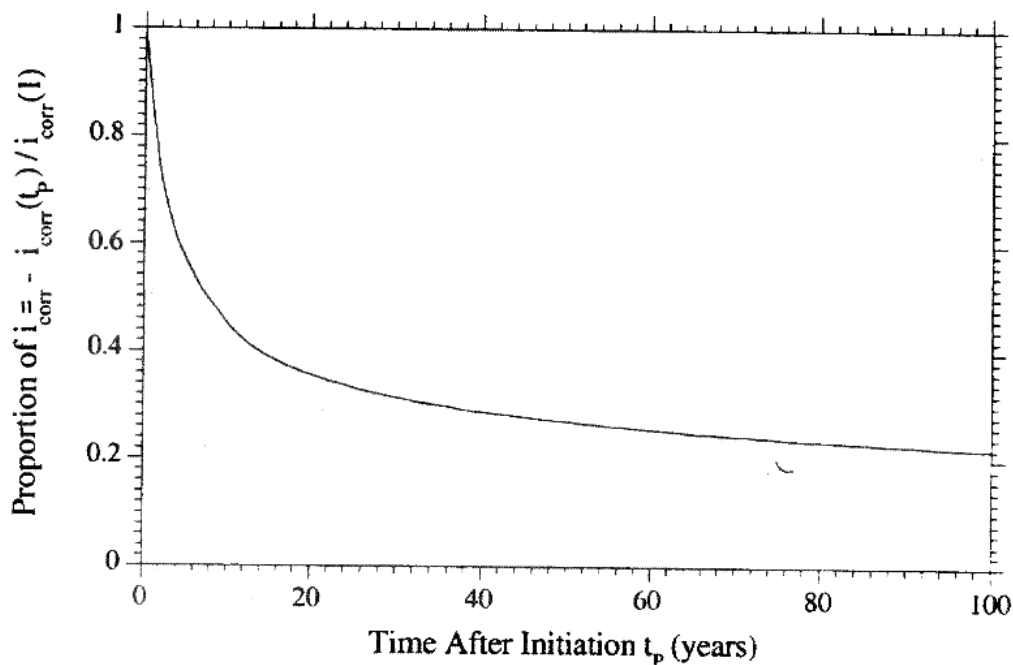


Figure 33. Effect of Time on Corrosion Rate

5.1.5 Best Fit Curves

Best-fit curves were developed to remove the unrelated “noise” of the system (Figures 34-38) using the first 6 months of data. All irregular data was deleted for the best-fit curve calculations. The uncharacteristic data resulted from power surges in the system or when the system was shut down to obtain half-cell readings. To determine the best-fit curve, it was assumed that the response curve was essentially exponential. Therefore, the natural log of the current was first calculated. The slope, y-intercept, and coefficient of correlation of the natural log of the corrosion current versus time were determined. The following equation was employed to derive the best-fit curve for each beam end:

$$y = ae^{-bt} \quad [\text{Eq. 5.1.5-1}]$$

Where, a is the exponent of the y-intercept (mAmps), b is the slope, and t is the time (hours). The coefficient of correlation was calculated for each curve. The coefficient of correlation expresses the strength of the linear relationship between the two variables. Hence a value of 1 indicates that the t and $\ln(y)$ are perfectly correlated. All curves possess a coefficient of correlation of 90% or greater. Therefore, it can be concluded that a linear relationship exists between the natural log of corrosion current and time, and the assumption of exponential curve is generally valid.

All curves demonstrated a decrease in corrosion current over time. As stated earlier, the pre-coated (west) end of beam 1 had a smaller corrosion current in comparison to the initially untreated end. The untreated (west) end of beam 2 also demonstrates a much smaller corrosion current than the other initially untreated (east) end of beam 2. Again, this is likely due to issues encountered with the electrical system, and is not representative of the actual corrosion damage occurring. The initially sealed (west) end of beam 3 had slightly less corrosion current at the start of the experiment, but the currents began to converge with the initially untreated (east) end of beam 3 at the end of the first phase of the exposure. This seems to indicate that over 6-month exposure to an accelerated corrosion regime, the pre-sealed end has slightly better effectiveness as applying no pretreatment.

Phase I Corrosion Current vs. Time - Beam 1

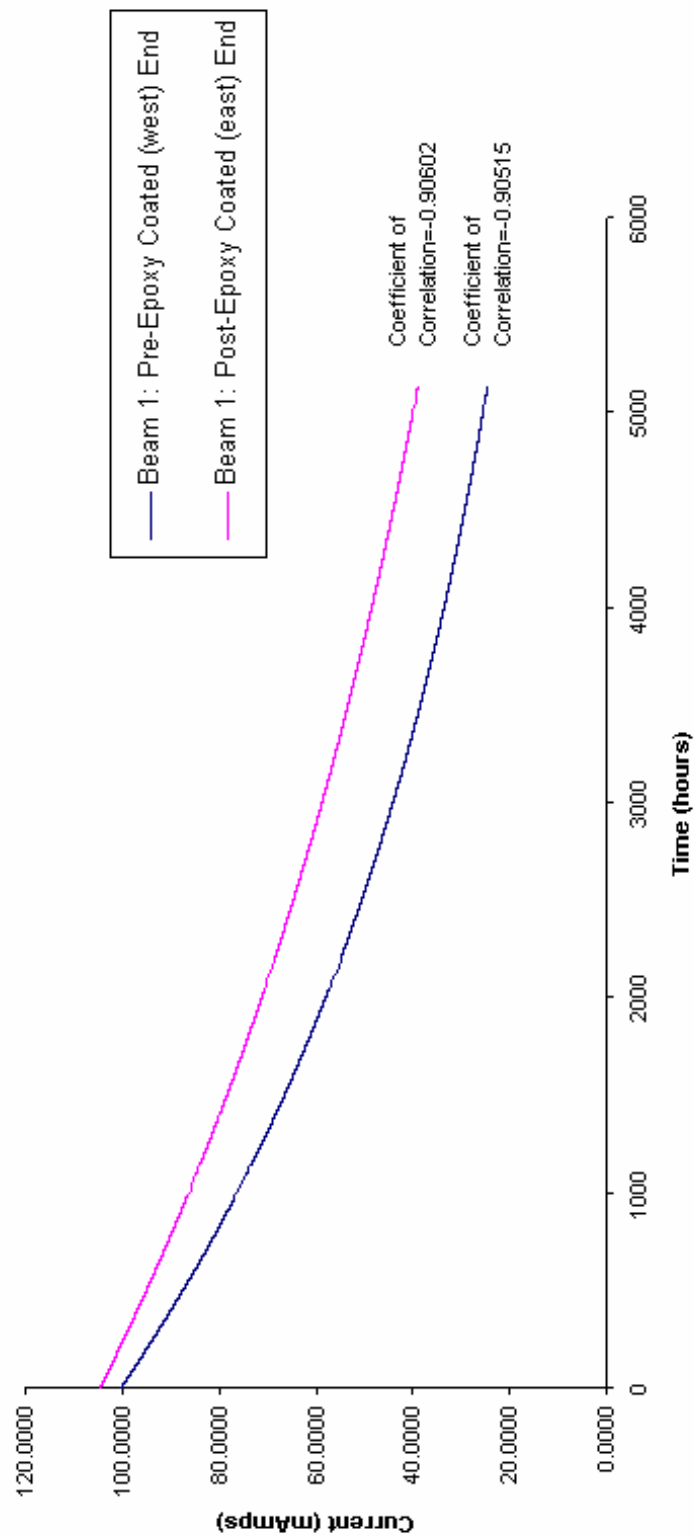


Figure 34. Best-Fit Curve: Corrosion Current vs. Time – Beam 1

Phase I Corrosion Current vs. Time - Beam 2

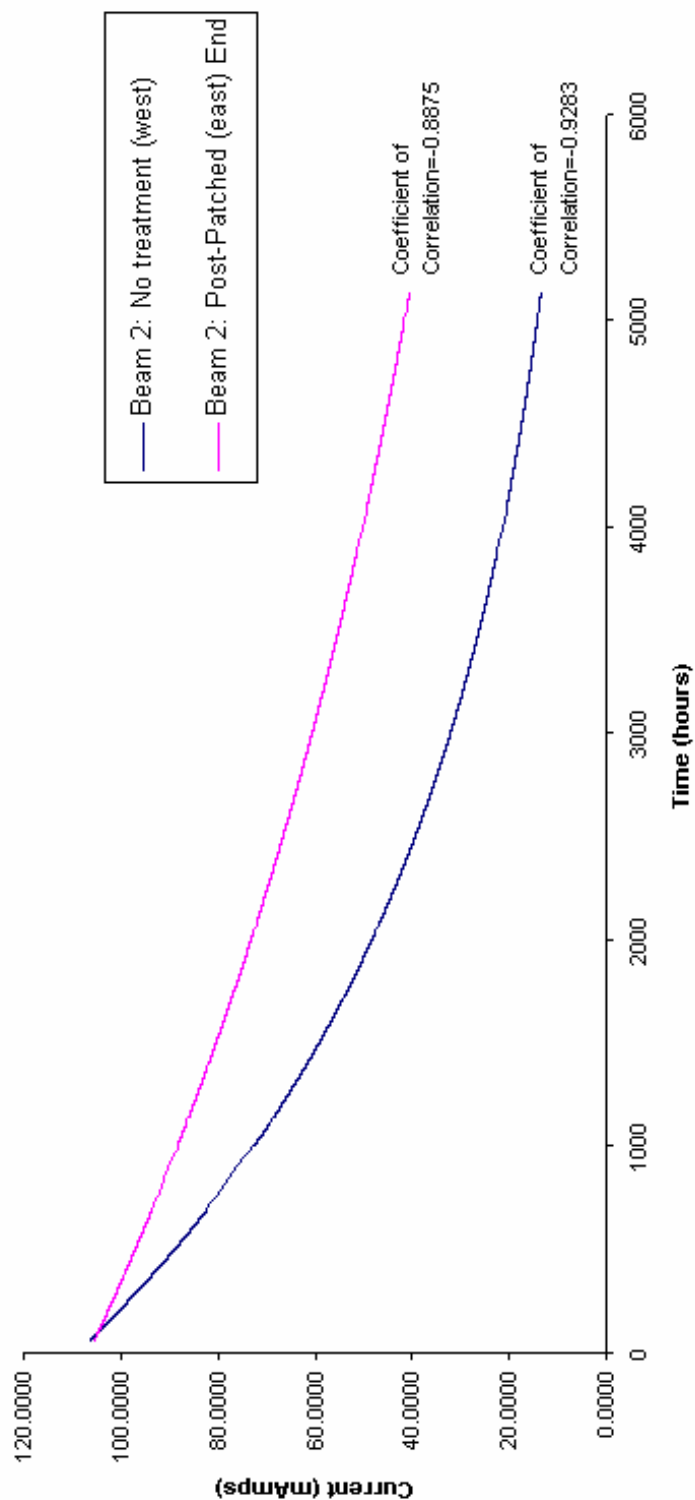


Figure 35. Best-Fit Curve: Corrosion Current vs. Time – Beam 2

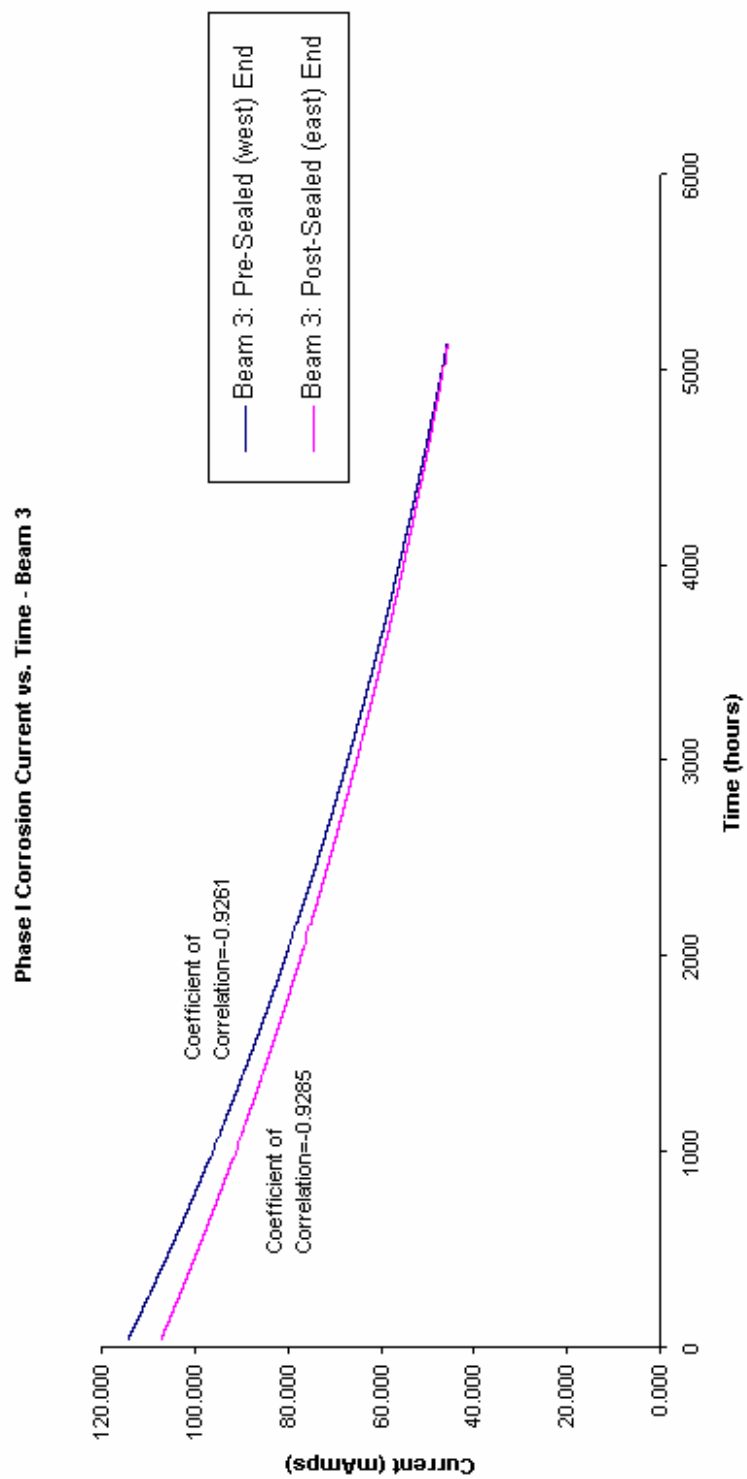


Figure .36. Best-Fit Curve: Corrosion Current vs. Time - Beam 3

Phase I Corrosion Current vs. Time - Beam 4

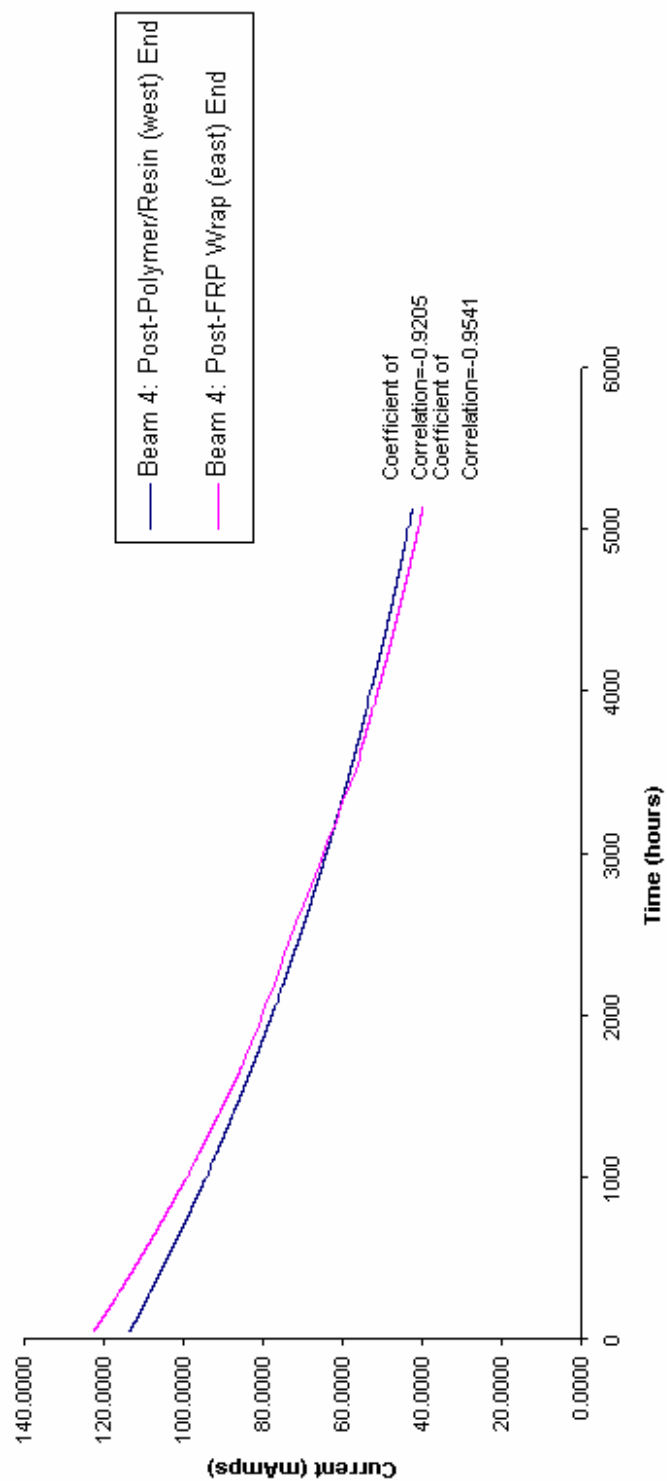


Figure 37. Best-Fit Curve: Corrosion Current vs. Time - Beam 4

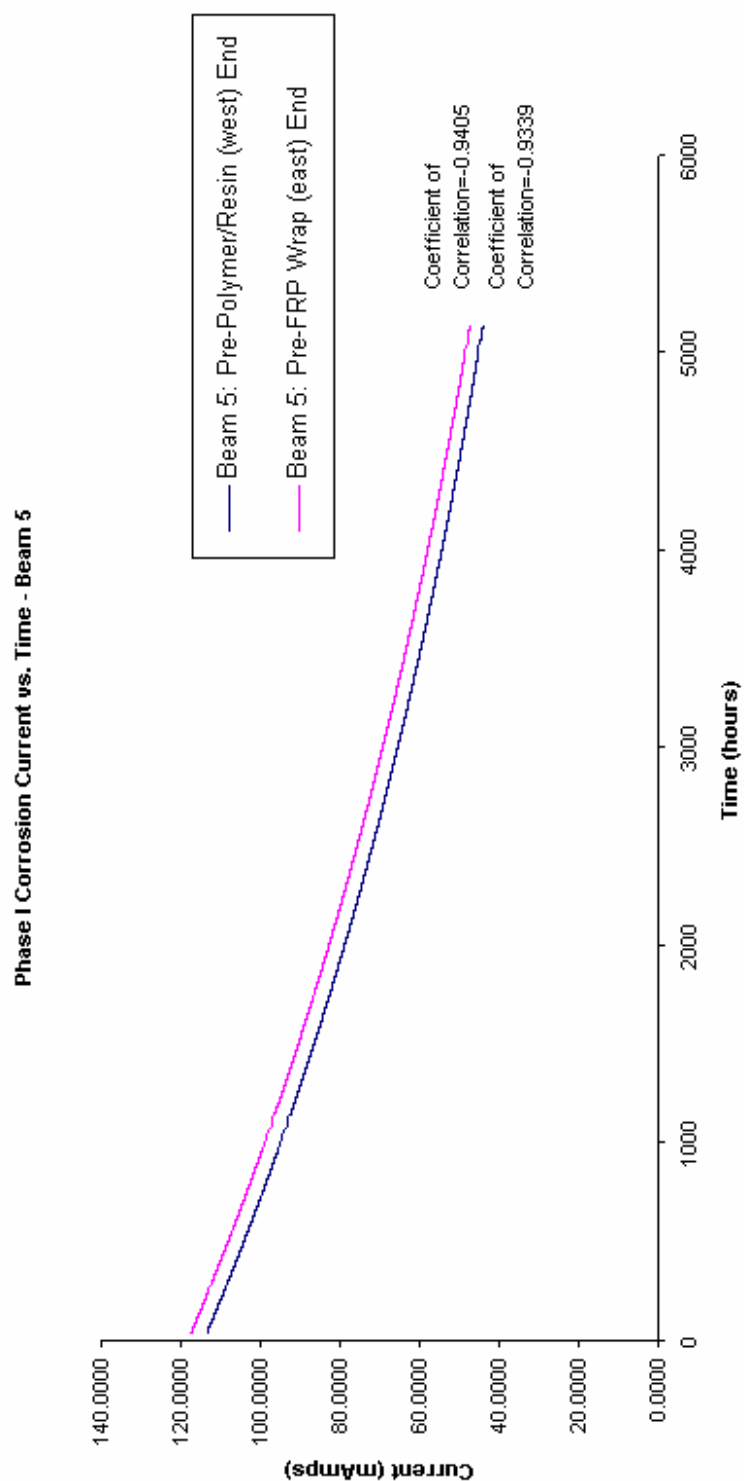


Figure 38. Best-Fit Curve: Corrosion Current vs. Time – Beam 5

Both initially untreated ends of beam 4 have nearly the same behavior for the first 6 months, which correlated with the fact that both ends were exposed to the same conditions. Both curves for beam 5 are decreasing at a very similar rate. The end pretreated with the polymer (resin) has a slightly less corrosion current in comparison to the end pretreated with the FRP wrap. Since the data for beam 5 are so similar, it can be deduced that both treatments have similar effectiveness at that time.

Figure 39 illustrates the combination of corrosion current versus time for all the beam-ends. The “no treatment” end of beam 2 exhibits the lowest corrosion current over time. As explained earlier, this is likely due to electrical connection problems with this particular end, and is not representative of its true behavior in preventing corrosion. The next lowest curve is the end of beam 1 pretreated with a coating. The end that exhibits the largest corrosion current versus time is the end pretreated with the FRP wrap. However, all curves for each of the beam-ends are clustered closely together. Therefore, based on the 6-month exposure data, a conclusive assessment of the effectiveness of various treatments cannot be made.

5.1.6 Steel Loss

Steel loss was estimated from the corrosion currents using the following equation:

$$w_t = \frac{At_m}{zF} \int I(t)dt \quad [\text{Eq. 5.1.6-1}]$$

where At_m is the atomic mass of the metal, z is its valency, F is Faraday’s constant (96487 C/mol), dt the time frame, and $I(t)$ is the best-fit curve extrapolated from the current

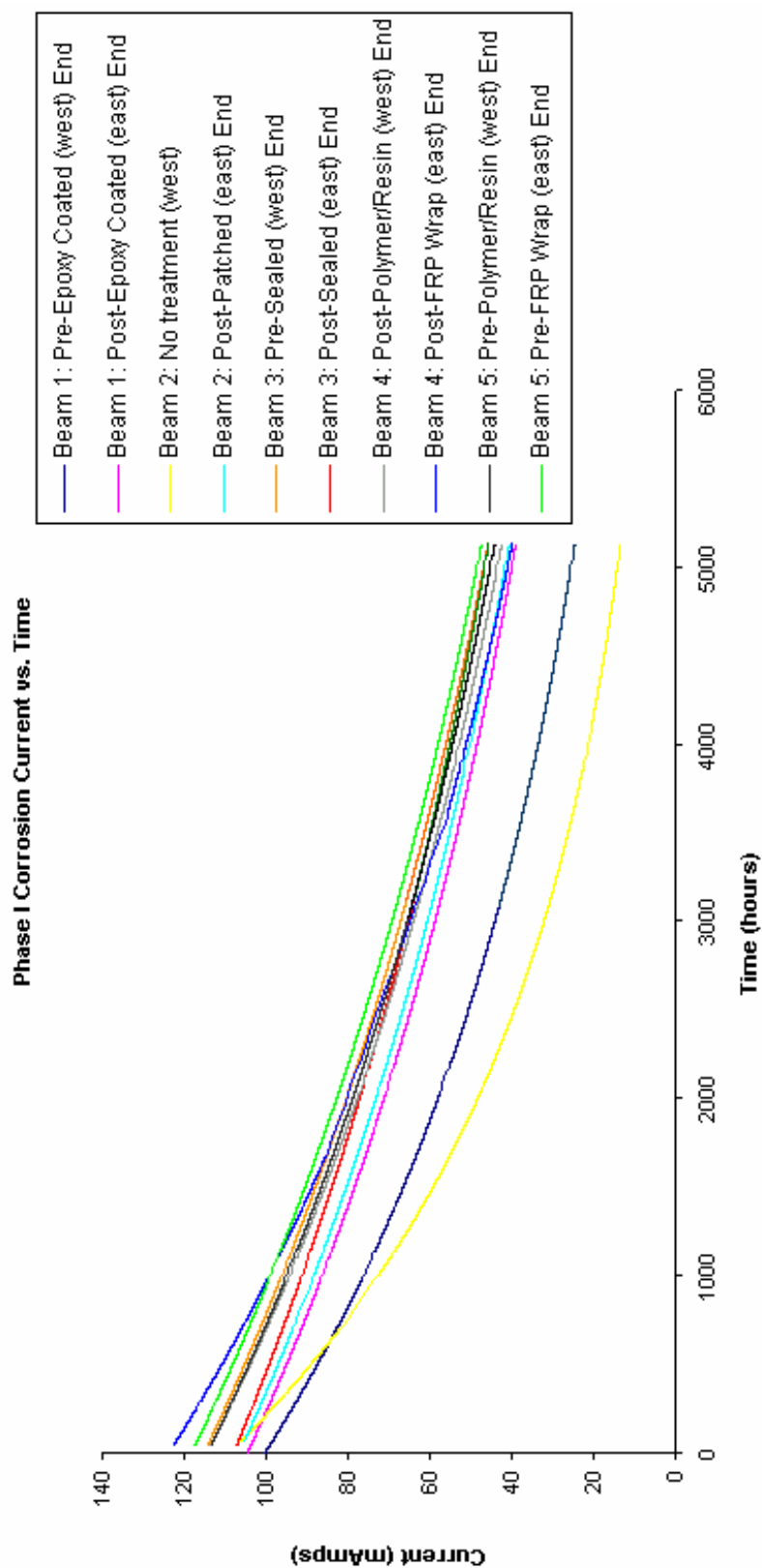


Figure 39. Best-Fit Curve: Corrosion Current vs. Time – All Beam Ends

measured. For reinforcing steel, which is primarily iron, the atomic mass is 55.85 g/mol and the valency is 2. Table 31 lists the calculated steel loss for each beam end (see Appendix E for calculations).

Table 31. Steel Loss

Beam End	Steel Loss	% Steel Loss**
Beam 1: pre-coated	288	1.7
Beam 1: untreated (post-coated)	356	2.1
Beam 2: untreated	245*	1.4*
Beam 2: untreated (patched)	366	2.1
Beam 3: pre-sealed	402	2.3
Beam 3: untreated (post-sealed)	387	2.2
Beam 4: untreated (post-polymer)	388	2.3
Beam 4: untreated (post-FRP)	396	2.3
Beam 5: pre-polymer	394	2.3
Beam 5: pre-FRP	415	2.4

*Possibly affected by electrical problem.

**Based on strand mass only (assuming corrosion takes place on strands only). These percentages could be reduced by applying a factor of 0.166 if the mass of the stirrups is considered.

The steel loss determined from the prior equation is calculated for the entire steel cage of the beam. The steel loss of interest is localized in the beam end regions and cannot be isolated from the loss over the entire reinforcing cage. Due to electrical problems, the value calculated for the untreated end of beam 2 is believed not to be accurate. It can be concluded that the pre-coated beam end (epoxy coated from Day 1) has experienced the least steel loss in comparison to the other ends. Also, the end pretreated with the FRP system polymer has experienced the highest steel loss. Not including the data from the “no treatment” end of beam 2, the average steel loss was 377 grams with a standard deviation of 38 grams. Since, the range of values is small, it cannot be conclusively determined which treatment provided the most effective corrosion protection from this method of analysis.

5.1.7 Half-Cell Potential Data

Half-cell measurements using a copper-copper sulfate electrode were obtained for each beam end. Half-cell measurements were taken approximately every month for the first exposure cycle. A contour plot of the half-cell readings at the beginning and end of the first exposure cycle as well as after 10 months of exposure are shown in Figures 42-58. As stated earlier, half-cell readings were not obtained for the treated beam-ends because of lack of electrical coupling in treated beams. The complete data and contour graphs are located in Appendix F.

Initial half-cell potentials were relatively uniform at all points measured. Whereas the half-cell readings after the first exposure cycle vary depending on their location on the beam. The values increase substantially as measurements neared the end of the beam. The highest readings were located on the bottom flange near the edge of the beam. These readings are consistent with the flow of the salt water down the end of the beam. The water normally traveled down the front face of the beam, curved around the bottom flange and then was collected in the trough system. Hence, the corrosion should be occurring in a similar location as the path of the salt water.

According to Emmons [15], it is generally agreed that the half-cell potential measurements can be interpreted as follows:

- Less negative than -0.20 volts indicates a 90% probability of no corrosion.
- Between -0.20 and -0.350 volts, corrosion activity is uncertain.

- More negative than -0.35 volts is indicative of a greater than 90% certainty that corrosion is occurring.

Since the polarity of the experimental setup is reversed from the standard method, the values obtained are in the positive range. According to Emmons [15] interpretation, after 6 months of exposure to a corrosive environment, the half-cell readings for the end of beam 1 indicate that corrosion is not occurring. The half-cell readings (at 6 months) for the ends of beam 2 and 4 indicate that it is inconclusive whether or not corrosion is occurring. However, the east end of Beam 3 has some half-cell readings on the bottom flange outer corner that indicate corrosion is occurring at these regions. The half-cell potential readings for beam ends 2A (no treatment) and 2B (patched) after 18 months of exposure clearly show corrosion activity in the beam-ends. Comparisons of Figure 45 with Figures 47 and 51, and Figure 46 with Figures 48 and 52 clearly show the progression of corrosion activities.

Half-cell potential readings were no longer taken on the surfaces treated after the first 6 months of exposure. The measurements were only taken on the non-treated beam-ends since surface treatments provide a non-conductive barrier that renders the half-cell measurements ineffective. The ends of beam 2 were the only remaining beam-ends that did not receive a surface treatment. After 10 months of exposure, the patched (east) end yields a higher potential in comparison to the untreated end. This trend is also observed at the end of 18 months. Figure 52 (patched end) shows a much larger area with half-cell potential readings of over 400 compared to Figure 51 (untreated end).

Figures 40 and 41 illustrate the orientation of the half-cell contour graphs on the prestressed beams, if directly facing the beam.

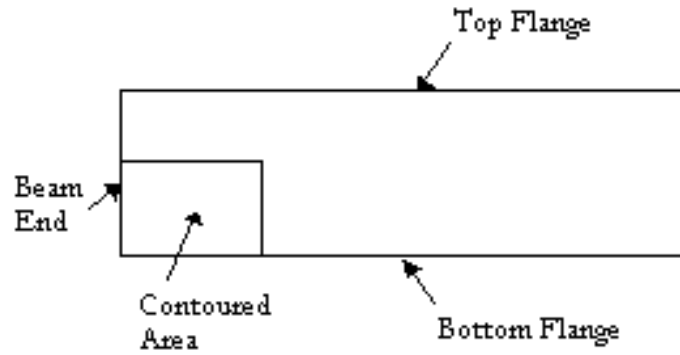


Figure 40. Southeast or Northwest Contour Orientation

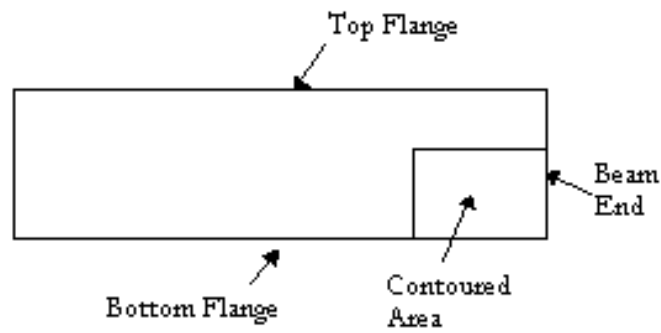


Figure 41. Southwest or Northeast Contour Orientation

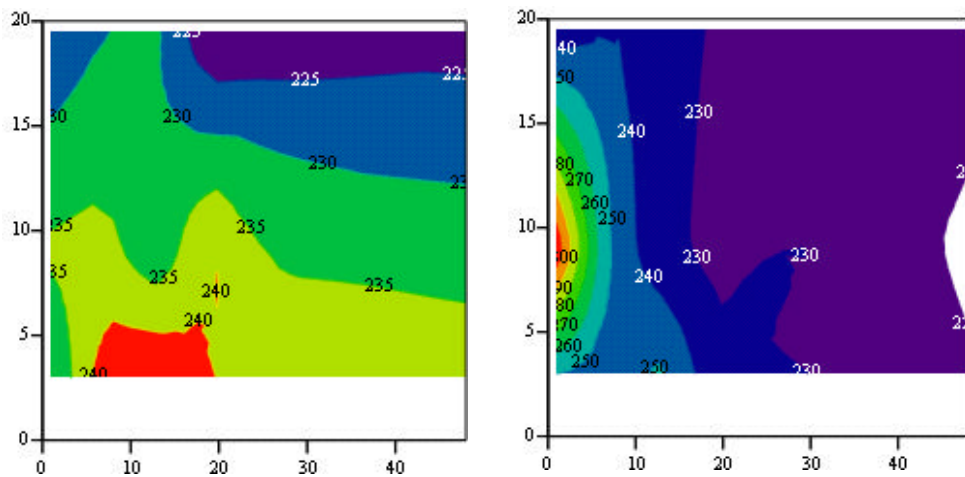


Figure 42. Initial Half-Cell Readings Beam 1B – Southeast End (left), Northeast End (right)

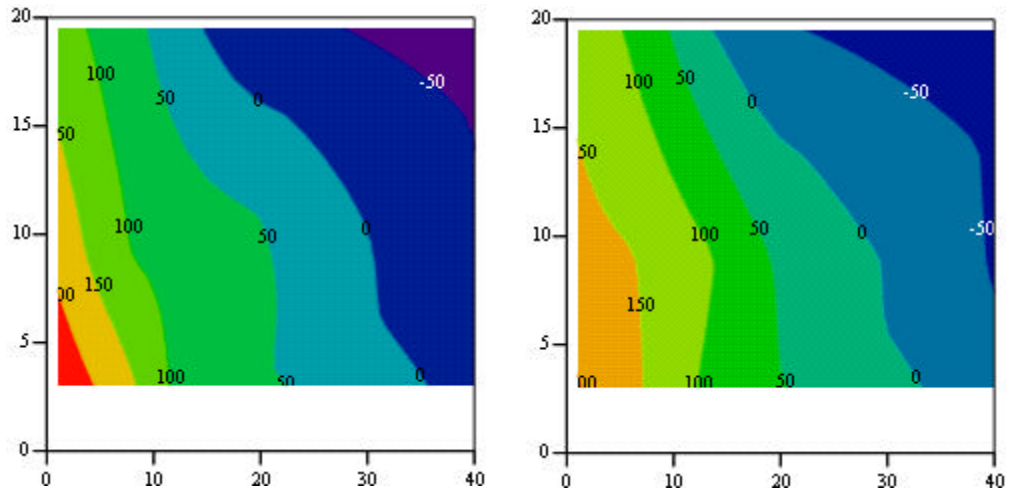


Figure 43. Half-Cell Readings Beam 1B(after 6 months) – Southeast End (left), Northeast End (right)

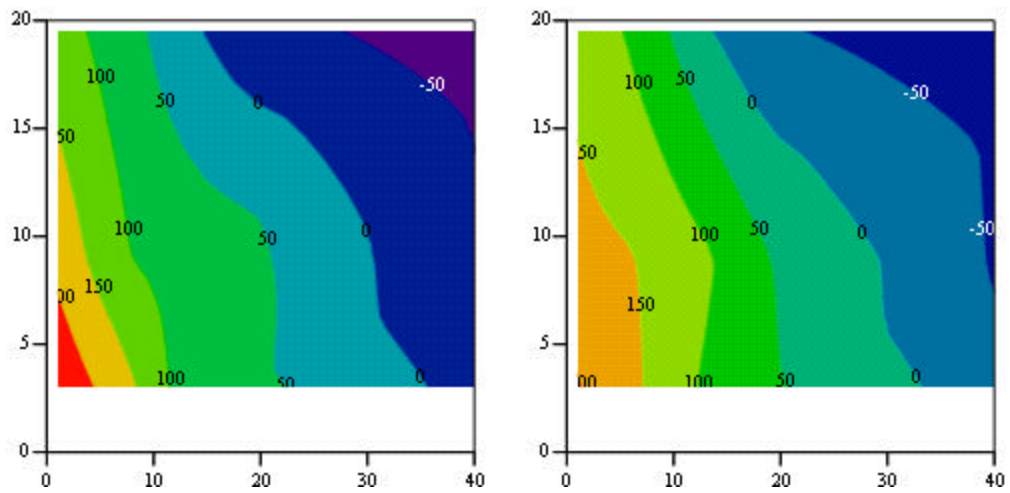


Figure 44. Half-Cell Readings Beam 1B(after 10 months) – Southeast End (left), Northeast End (right)

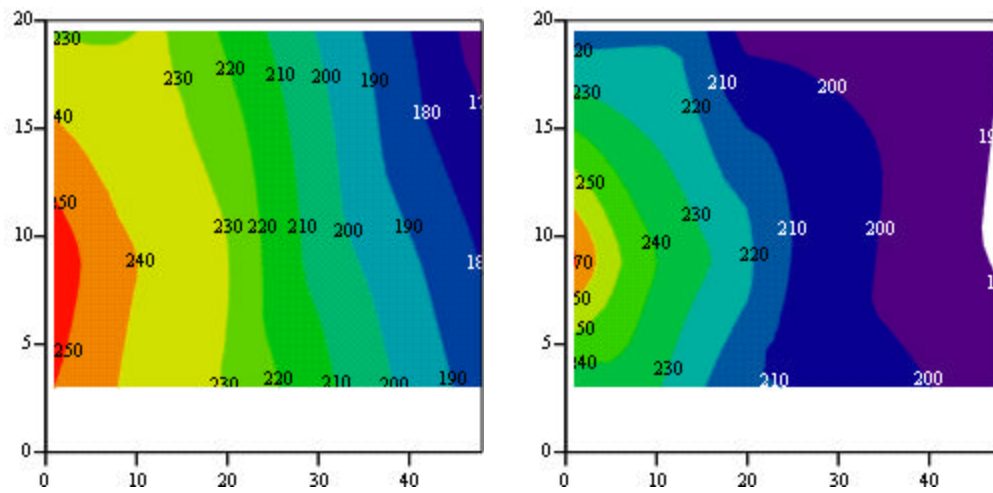


Figure 45. Initial Half-Cell Readings Beam 2A – Southwest End (left), Northwest End (right)

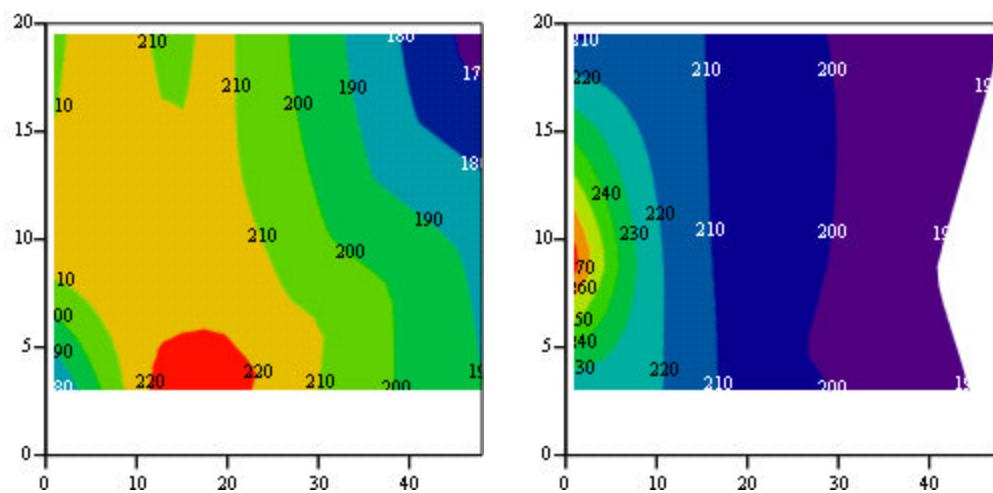


Figure 46. Initial Half-Cell Readings Beam 2B – Southeast End (left), Northeast End (right)

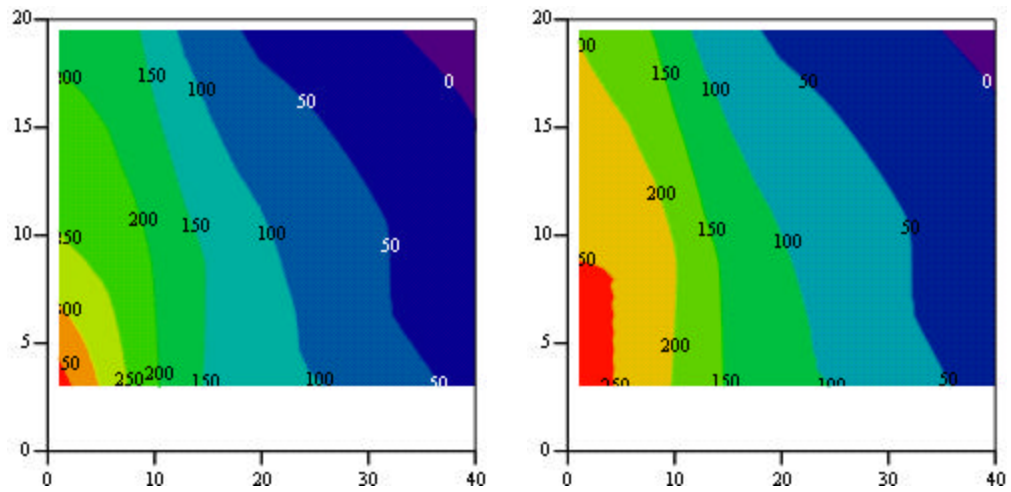


Figure 47. Half-Cell Readings Beam 2A(after 6 months) – Southwest End (left), Northwest End (right)

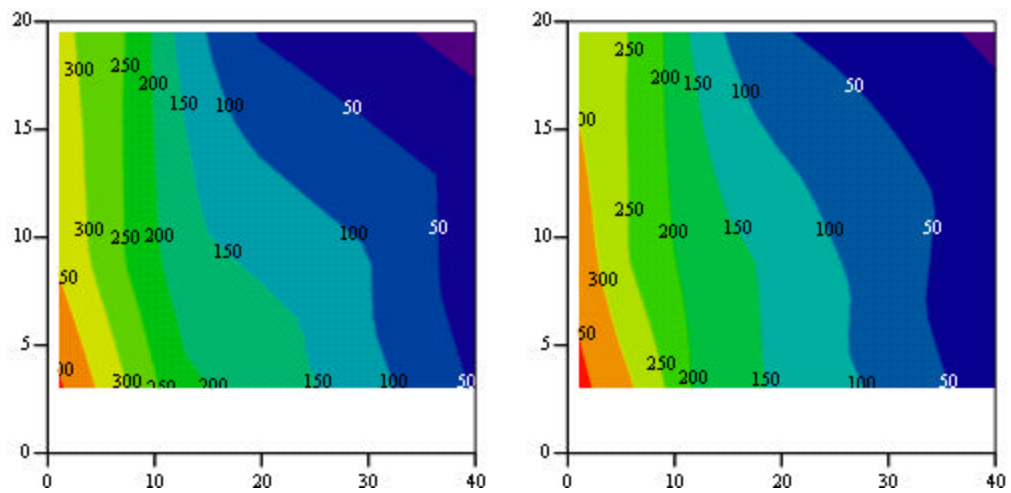


Figure 48. Half-Cell Readings Beam 2B(after 6 months) – Southeast End (left), Northeast End (right)

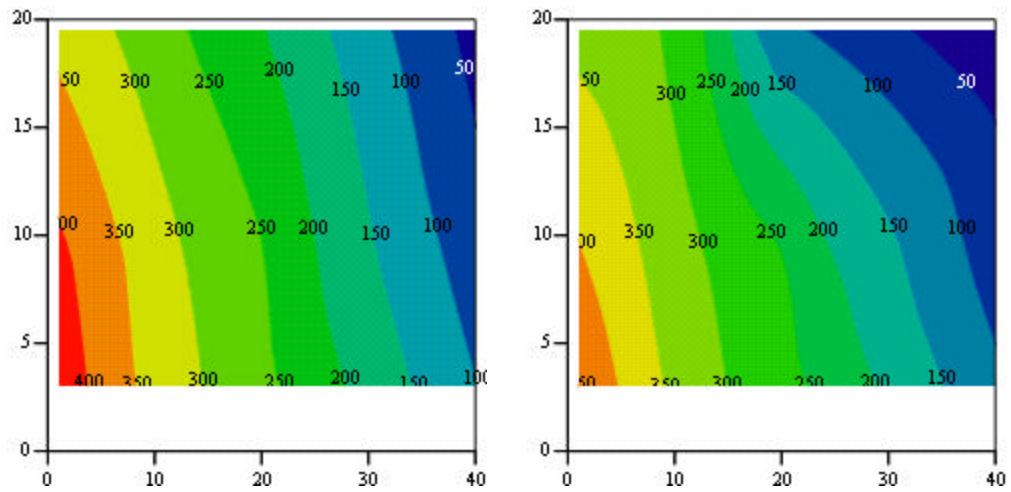


Figure 49. Half-Cell Readings Beam 2A(after 10 months) – Southwest End (left), Northwest End (right)

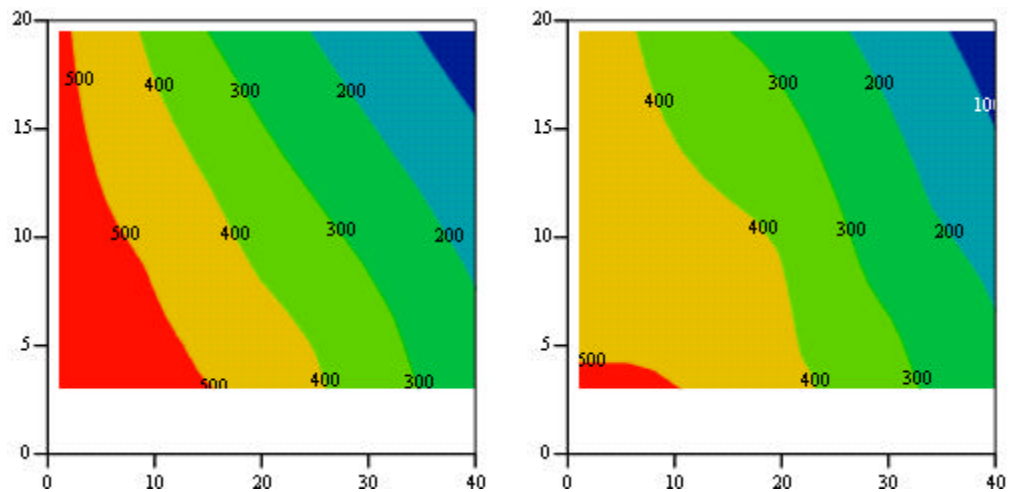


Figure 50. Half-Cell Readings Beam 2B(after 10 months) – Southeast End (left), Northeast End (right)

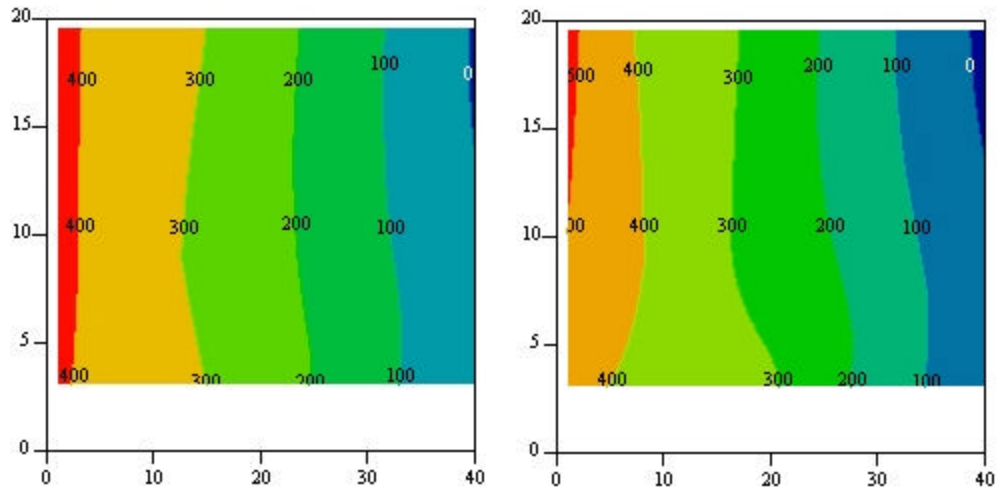


Figure 51. Half-Cell Readings Beam 2A(after 18 months) – Southwest End (left), Northwest End (right)

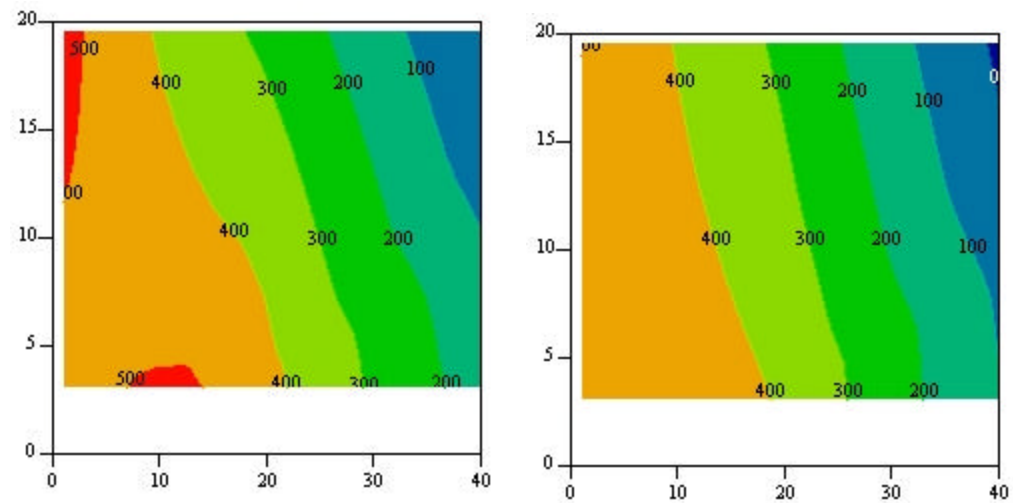


Figure 52. Half-Cell Readings Beam 2B(after 18 months) – Southeast End (left), Northeast End (right)

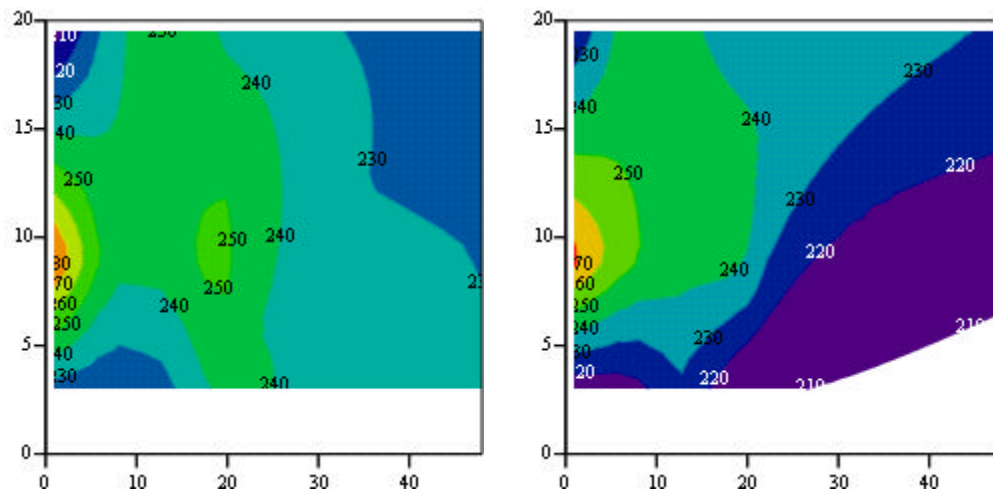


Figure 53. Initial Half-Cell Readings Beam 3B – Southeast End (left), Northeast End (right)

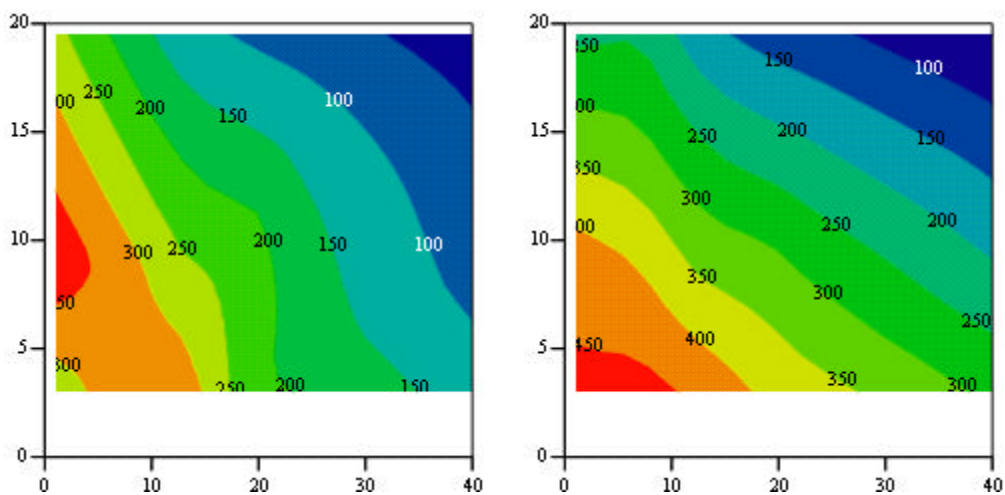


Figure 54. Half-Cell Readings Beam 3B(after 6 months) – Southeast End (left), Northeast End (right)

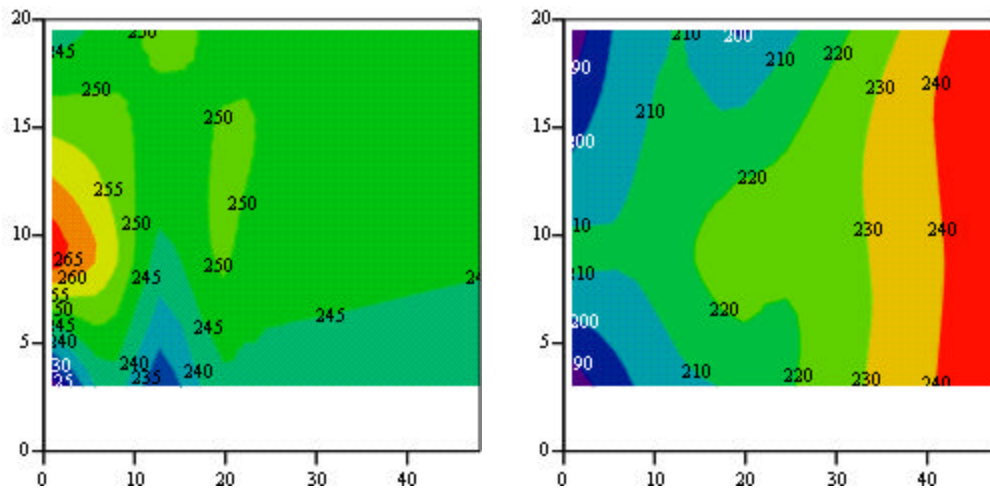


Figure 55. Initial Half-Cell Readings Beam 4A – Southwest End (left), Northwest End (right)

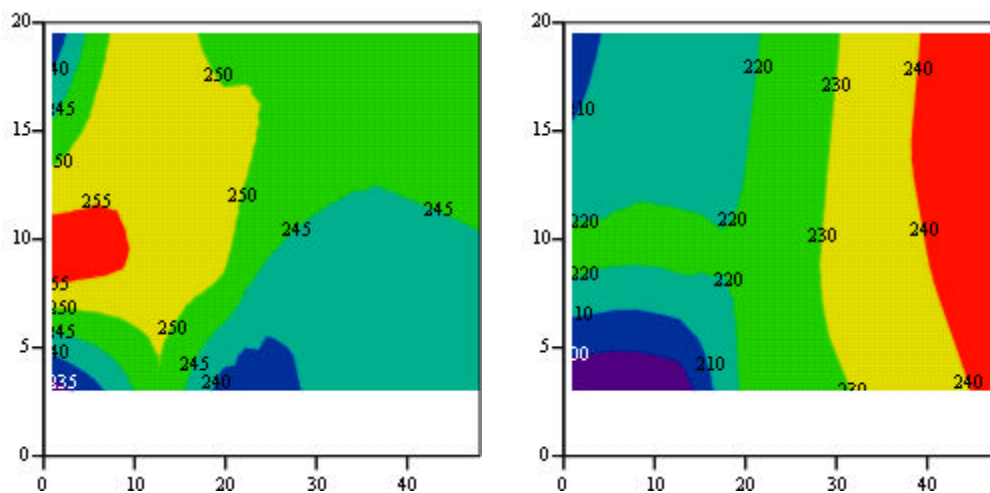


Figure 56. Initial Half-Cell Readings Beam 4B – Southeast End (left), Northeast End (right)

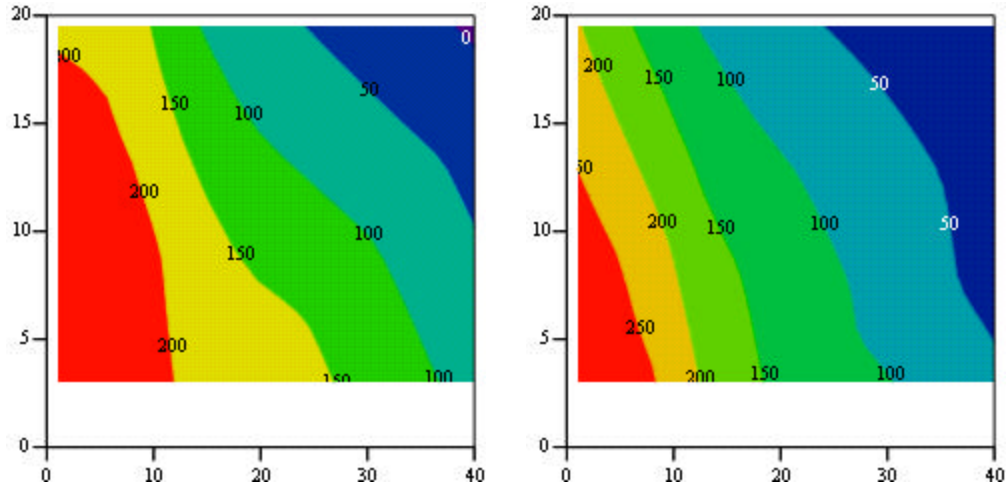


Figure 57. Half-Cell Readings Beam 4A(after 6 months) – Southwest End (left), Northwest End (right)

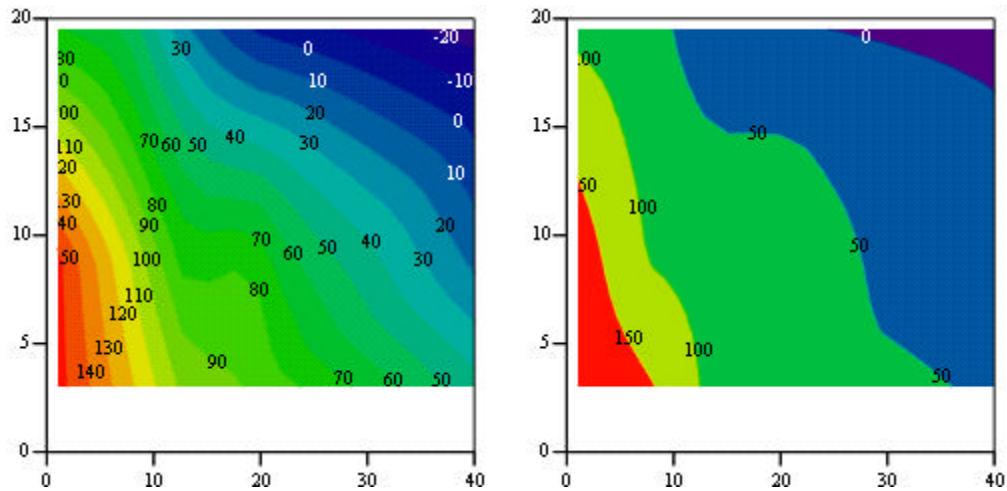


Figure 58. Half-Cell Readings Beam 4B(after 6 months) – Southeast End (left), Northeast End (right)

5.1.8 Strain Data

Displacement measurements using a mechanical displacement-measuring device were obtained for each beam end. Throughout the experiment, there were difficulties encountered keeping the points attached to the concrete surface. The brass points routinely would become loose, or become completely detached. Because of this, many of the ends do not have continuous

data over the course of the experiment. There were also difficulties encountered obtaining consistent readings with the measurement device. The tip of the device was conical, and hence the measurements could vary depending on the angle the device was placed into the point. Since many of the metal points were located in the path of the salt water, overtime they softened, with many corroding (see Figure 59).

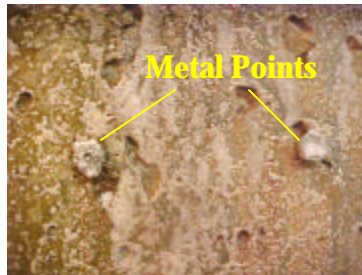


Figure 59. Measurement Points

Due to the softening of the metal, some of the “dimples” or depressions in the center of the points became warped. Hence, the readings would vary depending on where in the “dimple” the tip of the measuring device was placed. It was decided to not consider the gathered strain measurements due to the inconsistencies and inaccuracies of the data.

5.2 BEAM CONDITION OBSERVATIONS

5.2.1 Beam 1

Phase I – Beam 1

An epoxy coating was applied to the west end 1A (pre-coated) of beam 1 prior to the first accelerated corrosion cycle. The east end, 1B, remained untreated for the first exposure cycle. After 6 months, the untreated end (1B) was treated with the same epoxy coating applied to the

west end (1A) initially. Figure 60 illustrates the condition of 1A (pre-coated end) after 6 months of exposure. Figure 61 illustrates the condition of the southwest and northwest faces. Figure 62 illustrates the condition of the east end 1B after 6 months of exposure. Figure 63 illustrates the condition of the southeast and northeast faces.



Figure 60. Beam 1: West (pre-coated) Beam-End 1A (after 6 months)



Figure 61. Beam End 1A: Southwest Face (left), Northwest Face (right) (6 months)



Figure 62. Beam End 1B: East Beam-End, initially untreated (after 6 months)



Figure 63. Beam End 1B: Northeast Face (left), Southeast Face (right) (after 6 months)

At the completion of the first exposure cycle the beam-ends had heavy salt residue along the front faces and on some portions of the bottom flanges. Rust stains were also evident along the path of the salt water. No major spalling or cracking was observed. Some flaking of concrete was observed at the corners of the beam. In addition, corrosion products were observed on the exposed tendon ends, and were found to increase in amount over the course of the exposure.

Phase II – Beam 1

After 6 months of exposure, both beam-ends were cleaned to remove salt residue and rust products from the face of the beam. Then, a 2-foot section of the east end (1B) surface was

ground and thoroughly washed to remove all debris. The surface preparation and surface treatment application information was detailed in section 4.8. The east end (1B) was then treated with the epoxy coating. Figure 64 illustrates the condition of the west end (1A) after 18 months of exposure. Figure 65 illustrates the condition of the southwest side at the west end 1A. Figure 66 illustrates the condition of the east end face and southeast side of 1B (post-coated) after 18 months of exposure. Since the first 6-month exposure cycle did not result in the concrete spalling or significant tendon corrosion (section 5.1.4), the configuration of the saltwater dispersion system was altered slightly to increase the likelihood of corrosion after the second 6-month cycle. Pipes (1 foot long) were added along the north and south sides of each beam end to allow salt water to flow along the side face of the beams. The altered salt-water distribution setup was able to disperse water to both the sides and face of the beams. In addition, due to the new setup all of the beams were exposed to more water at a slightly faster flow rate.



Figure 64. Beam End 1A: West End (pre-epoxy coated)(after 18 months)



Figure 65. Beam End 1A: Southwest Face (after 18 months)



Figure 66. Beam 1: East End 1B (untreated, post-coated) (after 18 months)

5.2.2 Beam 2

Phase 1 – Beam 2

Both the west (2A) and east (2B) ends remained untreated for the first exposure cycle. After 6 months of exposure, portions of concrete were removed from the east end to facilitate installation of a patch repair. Figure 67 illustrates the condition of the west (untreated end 2A) after 6 months of exposure. Figure 68 illustrates the condition of the southwest face. Figure 69 illustrates the condition of the east (untreated end) after 6 months of exposure. Figure 70 illustrates the condition of the southeast and northeast faces.



Figure 67. Beam End 2A: West End (untreated)(After 6 months)



Figure 68. Beam End 2A: Northwest Face (left)(After 6 months)



Figure 69. Beam End 2B: East Face (untreated, patched)(After 6 months)



Figure 70. Beam End 2B: Northeast Face (left), Southeast Face (right)(After 6 months)

At completion of the first exposure cycle the beams-ends had heavy salt residue along the front faces and on some portions of the bottom flanges. Rust stains were also evident along the path of the salt water. No major spalling or cracking was observed. Some flaking of concrete was observed at the corners or edges of the beam. In addition, corrosion products were observed on the exposed tendon ends, and were found to increase in amount over the course of the exposure.

Phase II – Beam 2

After 6 months of exposure, both ends were cleaned to remove salt residue and rust products from the surface of the beam. Since 6 months of exposure did not result in the spalling of concrete, an 18-inch long concrete region of the east end was removed with a chipping hammer for installation of the patch repair (see Figure 71).



Figure 71. Beam Section Removed for Patch Repair

Figure 72 shows a close-up view of one of the strands. Corrosion products were observed mainly at the end regions of the strands. The build-up of corrosion products was seen to decrease as the distance from the edge of the end increased. The amount of corrosion was less than the researchers had expected. It was determined to change the configuration of the salt-water exposure to facilitate greater exposure on the sides of the beam to salt water.



Figure 72. Close-up View of Tendon from Dissected Beam End

Section 4.8 details the surface preparation and application methods used for the installation of the patch repair. Figure 73 shows the region after application of the bonding agent and after the patch repair material had been placed. The material was installed by a trowel, and pieces of lumber were used to shape the patched region. The region was allowed to cure according to the manufacturer's recommendation before it was re-exposed to the corrosive environment.



Figure 73. Application of Bonding Agent (left) and Patch Material (right)

Figure 74 illustrates the condition of the west (untreated) after 10 months of exposure. Figure 75 illustrates the condition of the southwest and northwest faces after 18 months of exposure.



Figure 74. Beam End 2A: West End (untreated) (After 10 months)



Figure 75. Beam End 2A: Southwest Face (left), Northwest Face (right) (After 18 months)

Figures 76 and 77 illustrate the condition of the southeast and northeast faces (patched) after 18 months of exposure.



Figure 76. Beam End 2B: Southeast Face (untreated, patched) (left), Northeast Face (right) (18 months)



Figure 77. Beam End 2B: Closer views (After 18 months)

After 10 months of corrosion exposure, the beam-ends of beam 2 were experiencing significant rust staining and salt residue. No spalling or major cracking was observed on the untreated (west) end. However, since the end was covered heavily in salt deposits, it was difficult to observe whether or not small hairline cracks were occurring. The patched (east) end experienced no major spalling, but cracks were observed in the patched region of the beam. Also, a vertical crack running the full height of the center section of the southeast end

approximately 3 inches from the edge was observed. Increased corrosion products were observed at all exposed steel tendon ends.

At the conclusion of testing (after 18 months of exposure), both ends of beam 2 had developed extensive cracking and corrosion stains were evident. All crack maps are shown in section 5.3 of this report.

5.2.3 Beam 3

Phase I – Beam 3

A sealer was applied to the west end (pre-sealed) of beam 3 prior to the first accelerated corrosion cycle. The east end remained untreated (post-sealed) for the first exposure cycle. After 6 months, the untreated end was treated with the same sealer applied to the west end initially. Figure 78 illustrates the condition of the west end 3A (pre-sealed) after 6 months of exposure. Figure 79 illustrates the condition of the southwest and northwest faces. Figure 80 illustrates the condition of the east end 3B (untreated post-sealed) after 6 months of exposure. Figure 81 illustrates the condition of the southeast and northeast faces.



Figure 78. Beam End 3A: West End (pre-sealed)(After 6 months)



Figure 79. Beam End 3A: Southwest Face (left), Northwest Face (right)(6 months)



Figure 80. Beam End 3B: East Face (untreated, sealed)(After 6 months)



Figure 81. Beam End 3B: Northeast Face (left), Southeast Face (right)(After 6 months)

At the completion of the first exposure cycle the beam-ends had heavy salt residue along the front faces and on some portions of the bottom flanges. Rust stains were also evident along

the path of the salt water. No major spalling or cracking was observed during the first six months of exposure. Some flaking of concrete was observed at the corners of the beam. In addition, corrosion products were observed on the exposed tendon ends, and were found to increase in amount over the course of the exposure.

Phase II – Beam 3

After 6 months of exposure, both beam-ends were cleaned to remove salt residue and rust products from the face of the beam. Then, a 2-foot section of the east end surface was ground and thoroughly washed to remove all debris. The surface preparation and surface treatment application information was detailed in section 4.8. The east end of beam 3 was then treated with the silane penetrating sealer. Figure 82 illustrates the condition of the west (pre-sealed end) after 10 months of exposure. Figure 83 illustrates the condition of the end face of 3A after 18 months of exposure. Figure 84 shows the condition of southwest face of beam end 3A. Figure 85 and 86 illustrate the condition of the beam end 3B (post-sealed) after 18 month.



Figure 82. Beam End 3A: West End (pre-sealed) (After 10 months)

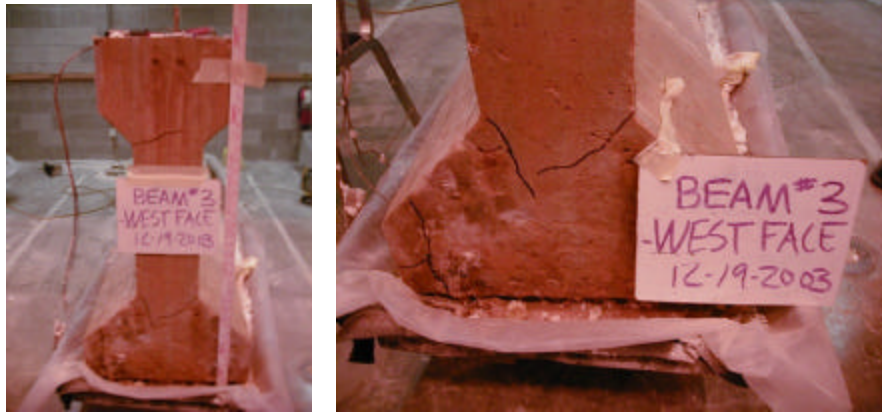


Figure 83. Beam End 3A: West End (pre-sealed)(After 18 months)



Figure 84. Beam End 3A: Southwest Face (After 18 months)



Figure 85. Beam End 3B: East End (After 18 months)



Figure 86. Beam End 3B: Northeast Face (left), Southeast Face (right) (After 18 months)

After approximately 10 months of exposure, the ends of beam 3 were experiencing significant rust staining and salt residue deposits. No spalling was observed on either of the beam-ends at that time. However, since the ends were covered heavily in salt deposits, it was difficult to observe whether or not small hairline cracks were occurring. Several cracks were observed on

the post-sealed (east) end of the beam. A large horizontal crack on the northeast bottom flange, about 10 inches in length, was observed. Also, a vertical crack on the bottom flange of the east face was observed.

At the conclusion of testing (after 18 months of exposure), both ends of beam 3 had developed extensive cracking and corrosion stains were evident. All crack maps are shown in section 5.3 of this report.

5.2.4 Beam 4

Phase I – Beam 4

Both the west and east ends remained untreated for the first exposure cycle. After 6 months of exposure, the FRP system was applied to the east end and the polymer (resin) was applied to the west end of the beam. Figure 87 illustrates the condition of the west (untreated end) after 6 months of exposure. Figure 88 illustrates the condition of the southwest and northwest faces. Figure 89 illustrates the condition of the east (untreated end) after 6 months of exposure. Figure 90 illustrates the condition of the southeast and northeast faces.



Figure 87. Beam End 4A: West End (untreated, post-polymer)(After 6 months)



Figure 88. Beam End 4A: Southwest Face (left), Northwest Face (right)(After 6 months)



Figure 89. Beam End 4B: East Face (untreated, post-FRP)(After 6 months)



Figure 90. Beam End 4B: Northeast Face (left), Southeast Face (right)(After 6 months)

At completion of the first exposure cycle the beams-ends had heavy salt residue along the front faces and on some portions of the bottom flanges. Rust stains were also evident along

the path of the salt water. No major spalling or cracking was observed. Some flaking of concrete was observed at the corners or edges of the beam. In addition, corrosion products were observed on the exposed tendon ends, and were found to increase in amount over the course of the exposure.

Phase II – Beam 4

After 6 months of exposure, both beam-ends were cleaned to remove salt residue and rust products from the face of the beam. Then, a 2-foot section of both end surfaces were ground and thoroughly washed to remove all debris. The surface preparation and surface treatment application information was detailed in section 4.8. The east end was then treated with the FRP system and the west end was treated with the polymer (resin). Figure 91 illustrates the condition of the west end 4A after 10 months of exposure. Figure 92 illustrates the condition of the west end 4A after 18 months of exposure. Figure 93 illustrates the condition of the east end 4B (post FRP) after 18 months of exposure.



Figure 91. Beam End 4A: West End (untreated, post-polymer) (After 10 months)



Figure 92. Beam End 4A: Southwest Face (left), Northwest Face (right) (After 18 months)



Figure 93. Beam End 4B: Northeast (untreated, post-FRP) (left), Southeast (right) (18 months)

No spalling or major cracking was observed on either of the beam-ends at the end of 18 months of exposure.

5.2.5 Beam 5

Phase I – Beam 5

A polymer (resin) was applied to the west end 5A (pre-polymer) of beam 5 prior to the first accelerated corrosion cycle. In addition, the FRP system was applied to the east end 5B (pre-FRP) of beam 5 before the first exposure cycle. Figure 94 illustrates the condition of the west end 5A (pre-polymer end) after 6 months of exposure. Figure 95 illustrates the condition of the southwest and northwest faces. Figure 96 illustrates the condition of the east end 5B (post-FRP end) after 6 months of exposure. Figure 97 illustrates the condition of the southeast and northeast faces.



Figure 94. Beam End 5A: West End (pre-polymer)(After 6 months)



Figure 95. Beam End 5A: Southwest Face (left), Northwest Face (right)(After 6 months)



Figure 96. Beam End 5B: East Face (pre-FRP)(After 6 months)

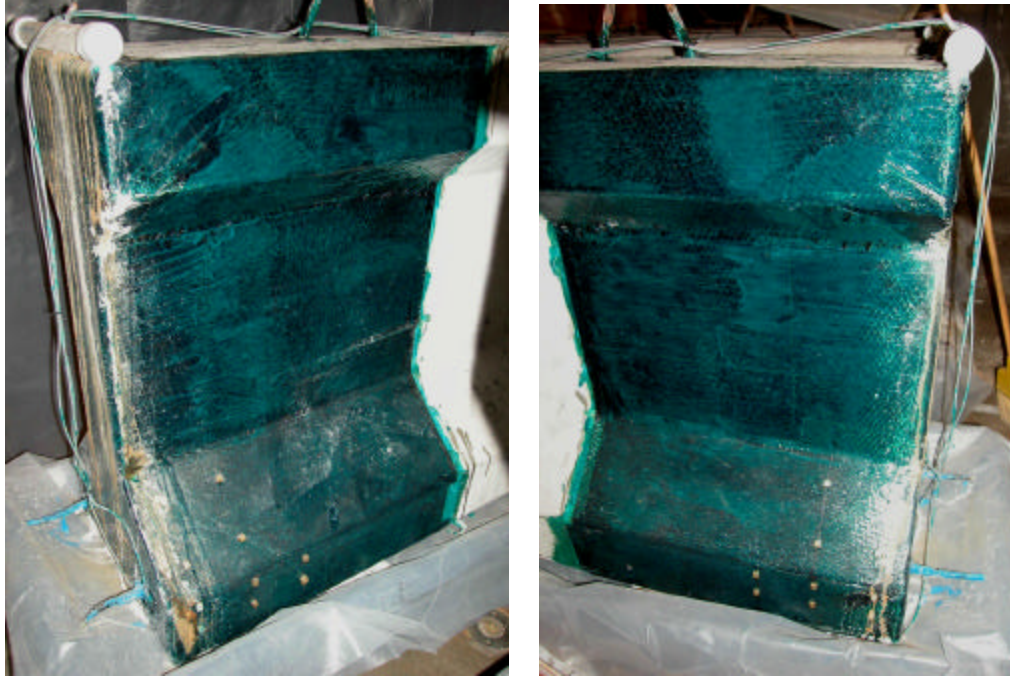


Figure 97. Beam End 5B: Northeast Face (left), Southeast Face (right)(After 6 months)

No major spalling or cracking was observed at the end of 6 months of exposure.

Phase II – Beam 5

After 6 months of exposure, both beam-ends were cleaned to remove salt residue and rust products from the face of the beam. No additional actions were taken for both beam-ends. Figure 98 illustrates the condition of the west (pre-polymer end) after 10 months of exposure. Figure 99 illustrates the condition of the southwest and northwest faces of beam 5 after 18 months of exposure. Figure 100 illustrates the condition of the southeast and northeast (pre-FRP) faces after 18 months of exposure.



Figure 98. Beam End 5A: West End (pre-polymer) (After 10 months)



Figure 99. Beam End 5A: Southwest Face (left), Northwest Face (right) (18 months)



Figure 100. Beam End 5B: Northeast Face (pre-FRP) (left), Southeast Face (right) (18 months)

After 18 months of exposure, no spalling or major cracking was observed on either of the beam-ends. Crack maps for all beams are shown in section 5.3.

5.3 CRACK MAPS

5.3.1 Beam 1

Crack maps for all beam-ends were obtained at the completion of the first 6 months of exposure and at the conclusion of all tests (18 months). Figures 101 and 102 illustrate the crack maps for the west (1A, pre-coated) and east (1B, untreated, post-coated) ends. The crack widths on the west face varied from 0.005 to 0.010 inches. The northwest face crack width was 0.005 inches and the southwest face crack width was 0.010 inches. The crack width on the east face was measured to be equal to 0.005 inches. No cracks were observed on the northeast and southeast faces.

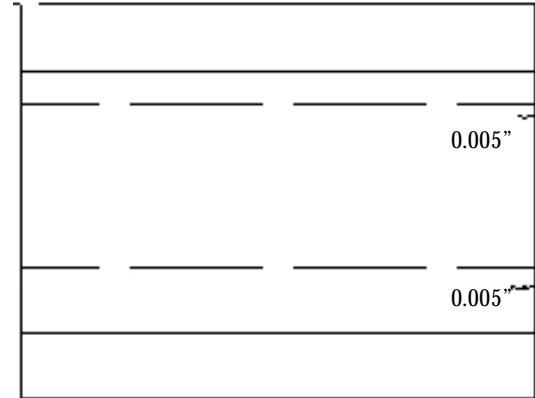
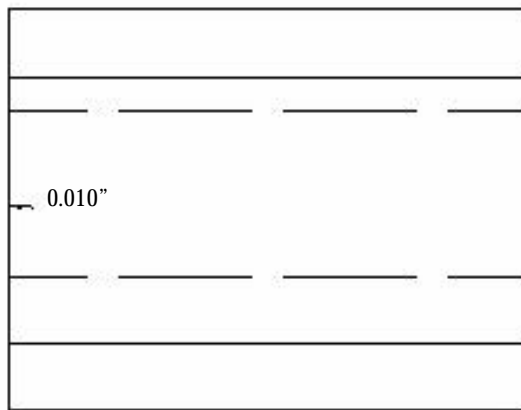
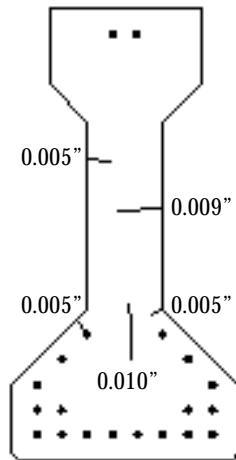


Figure 101. Beam End 1A: West End (top), Southwest (left), Northwest (right) (6 months)

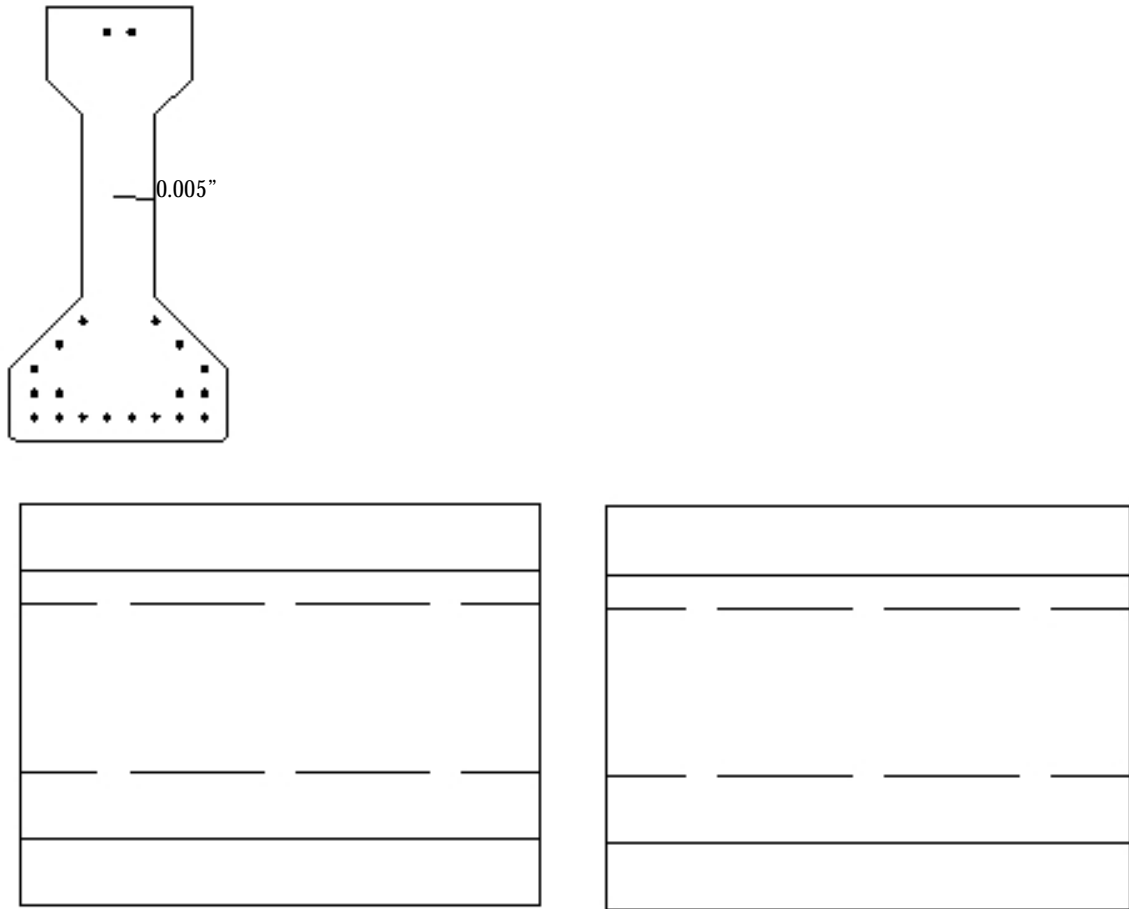


Figure 102. Beam End 1B: East End (top), Southeast Face (left), Northeast Face (right) (6 months)

Figures 103 and 104 show crack maps for the 1A and 1B ends at the conclusion of testing (18 month). There is only a slight progression of cracking on the northwest side of 1A from 6 months of exposure to 18 months. However, the 1B end did develop extensive new cracks at the end of the 18-month exposure period. This is expected as 1B was subjected to 6 months of unprotected exposure to saltwater before application of epoxy coating.

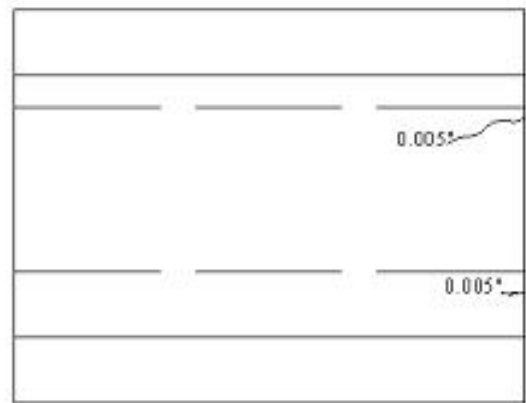
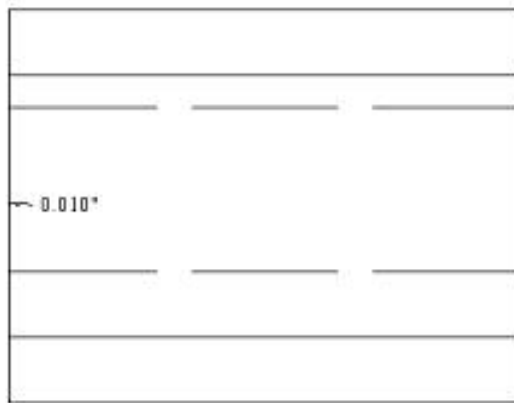
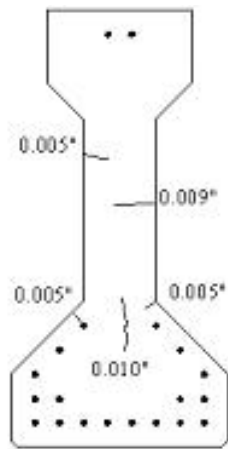


Figure 103. Beam End 1A: West End (top), Southwest (left), Northwest (right) (18 months)

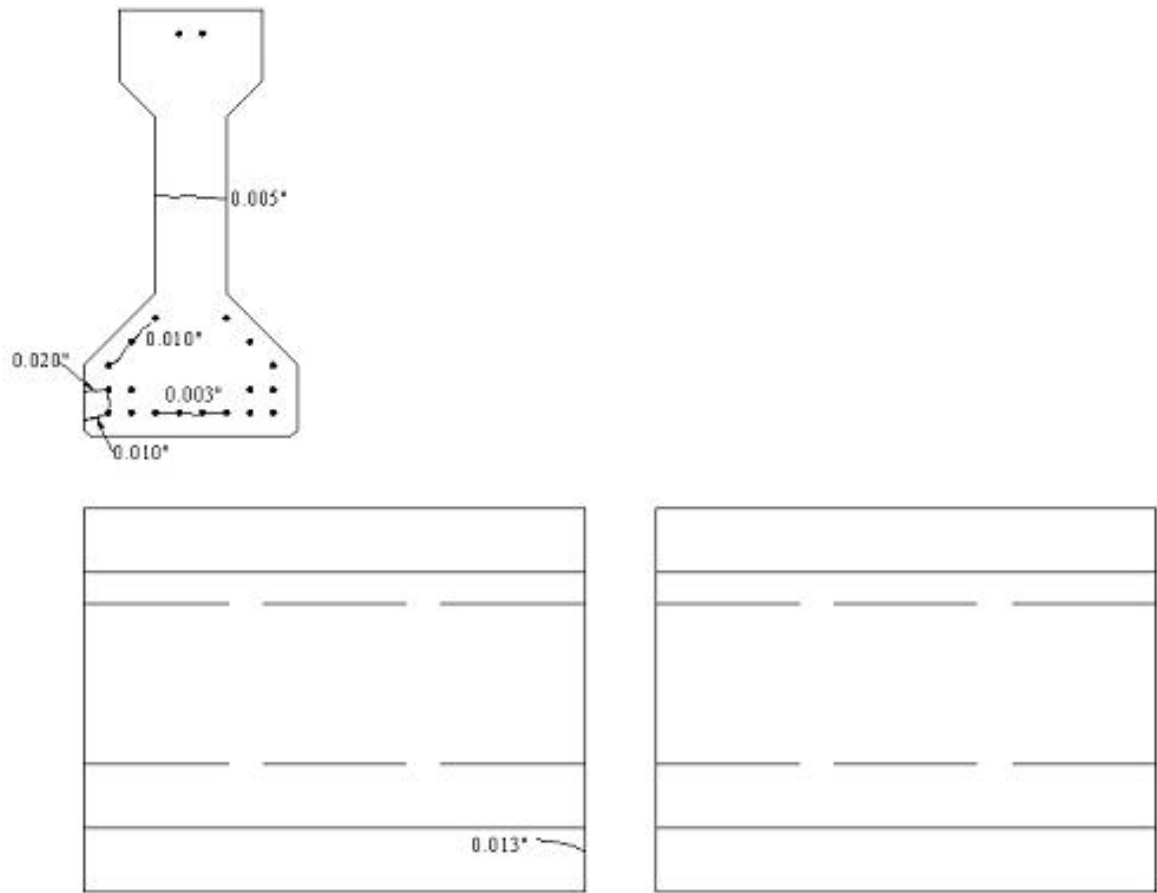


Figure 104. Beam End 1B: East End (top), Southeast (left), Northeast (right) (18 months)

5.3.2 Beam 2

Crack maps for both beam-ends were obtained at the completion of the first 6 months of exposure and at the conclusion of all tests. Figures 105 and 106 illustrate the crack maps for the west end 2A (untreated) and east end 2B (untreated, patched) at the end of 6 months of exposure. The crack widths on the west face varied from 0.003 to 0.005 inches. The northwest face crack width was 0.002 inches and the southwest face crack width was 0.002 inches. The crack widths on the east face ranged between 0.002 and 0.003 inches. The southeast crack widths were between 0.002 and 0.003 inches. No cracks were observed on the

northeast face. Crack maps after 18 months of exposure for beam ends 1A and 1B are shown in figures 107 and 108, respectively.

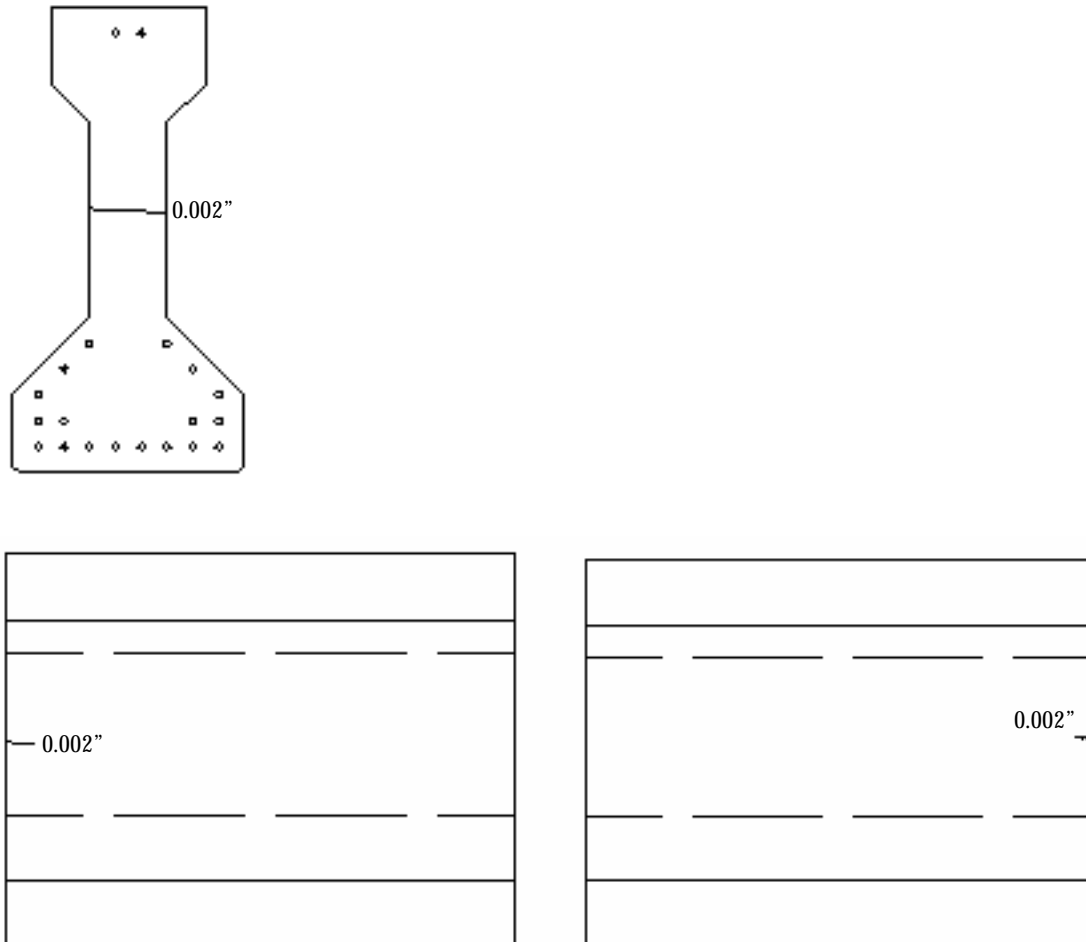


Figure 105. Beam End 2A: West End (top), Southwest (left), Northwest (right)(6 months)

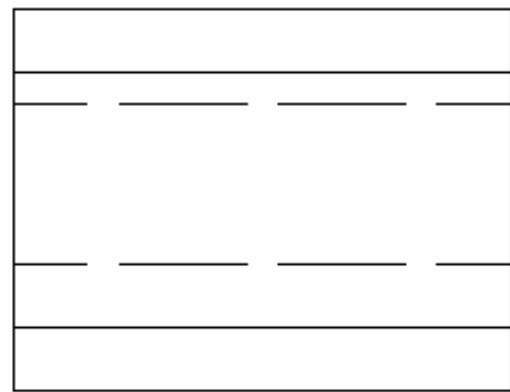
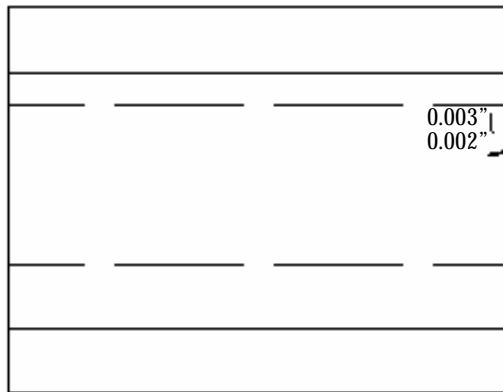
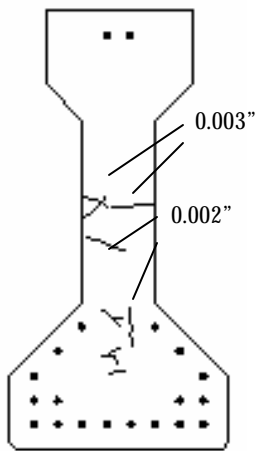


Figure 106. Beam End 2B: East End (top), Southeast (left), Northeast (right) (6 months)

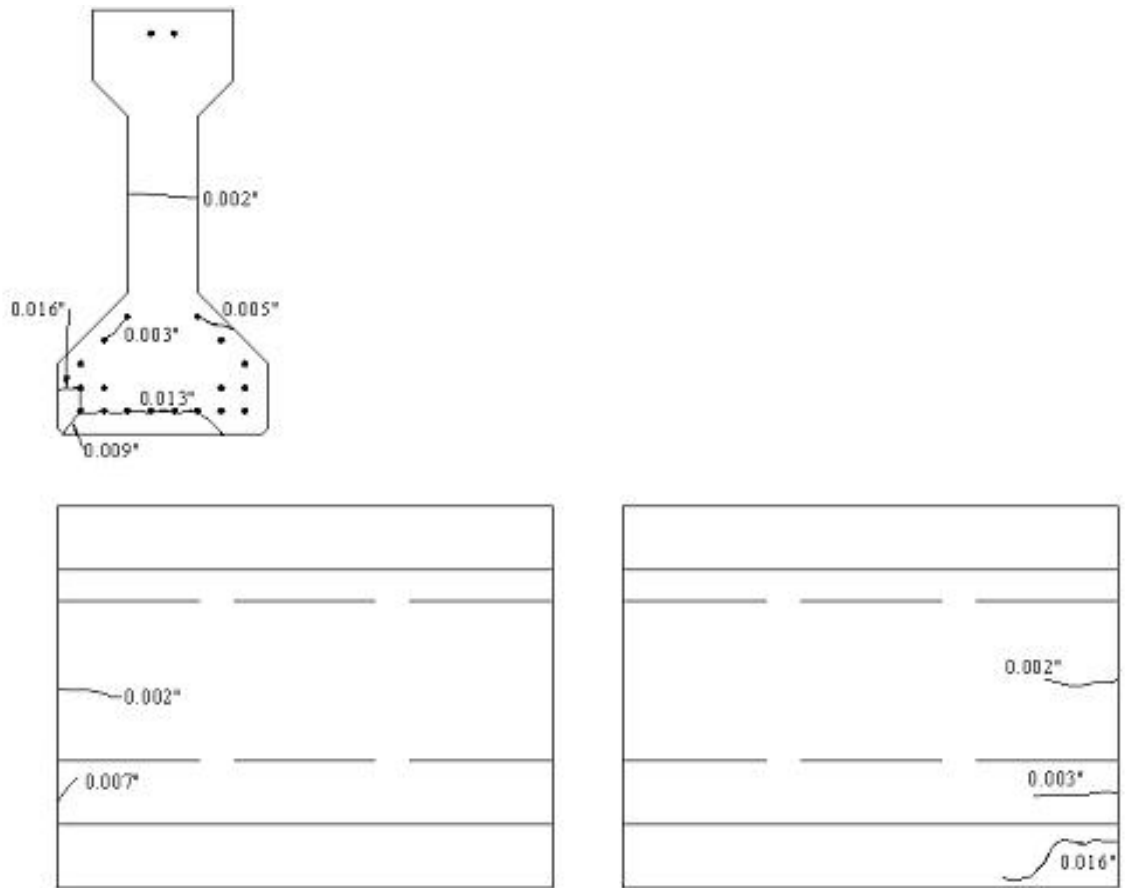


Figure 107. Beam End 2A: West End (top), Southwest (left), Northwest (right)(18 months)

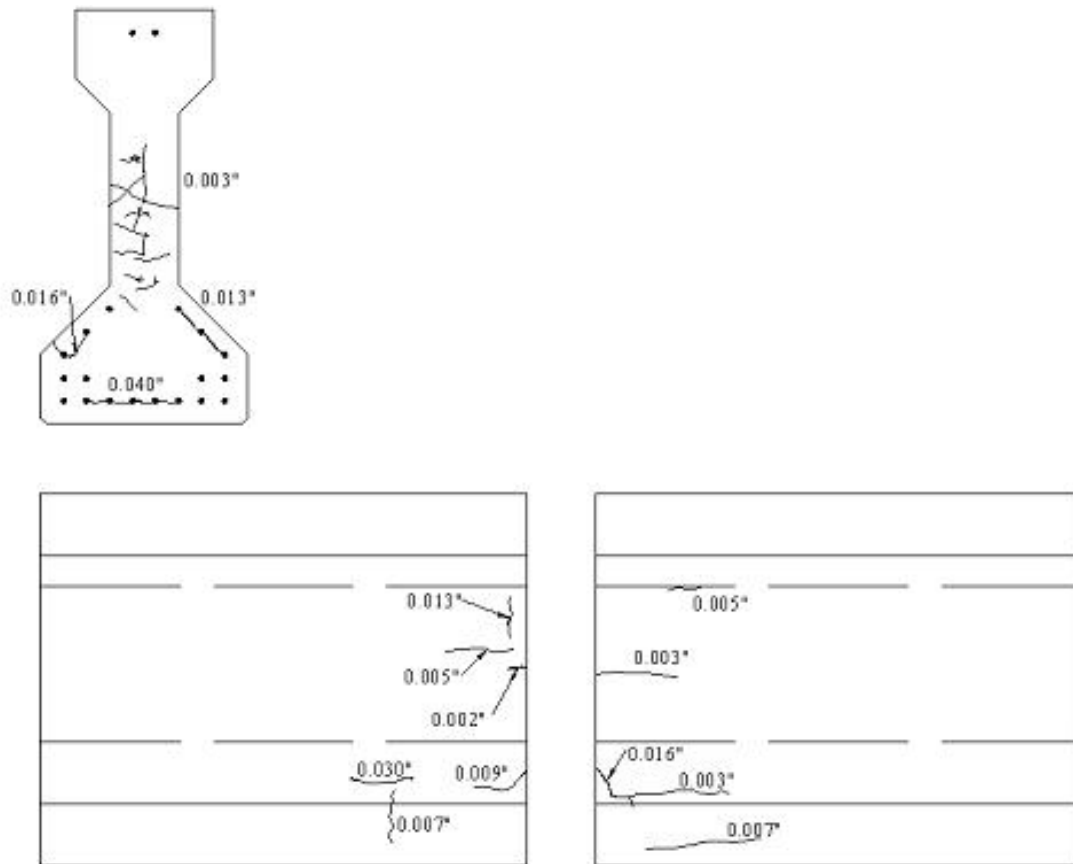


Figure 108. Beam End 2B: East End (top), Southeast (left), Northeast (right)(18 months)

It is clear that cracking grew between 6 and 18 months, especially for beam end 2B (patched end). The cracks between strands indicate the onset of spalling. The crack sizes are also large.

5.3.3 Beam 3

Crack maps for both beam-ends were obtained at the completion of the first 6 months of exposure and the conclusion of all tests. Figures 109 and 110 illustrate the crack maps for the west end 3A (pre-sealed) and east end 3B (untreated, post-sealed). After 6 months, the crack widths on the west face varied from 0.003 to 0.005 inches. The northwest face crack widths ranged between 0.003 and 0.010 inches and the southwest face crack widths were between

0.002 and 0.020 inches. The crack widths on the east face ranged between 0.002 and 0.010 inches. The northeast crack widths were between 0.002 and 0.005 inches. No cracks were observed on the southeast face after 6 months of exposure. Figure 111 and 112 show crack maps after 18 months of exposure for 3A and 3B, respectively. Again, crack growth was clearly evident in both beam-ends.

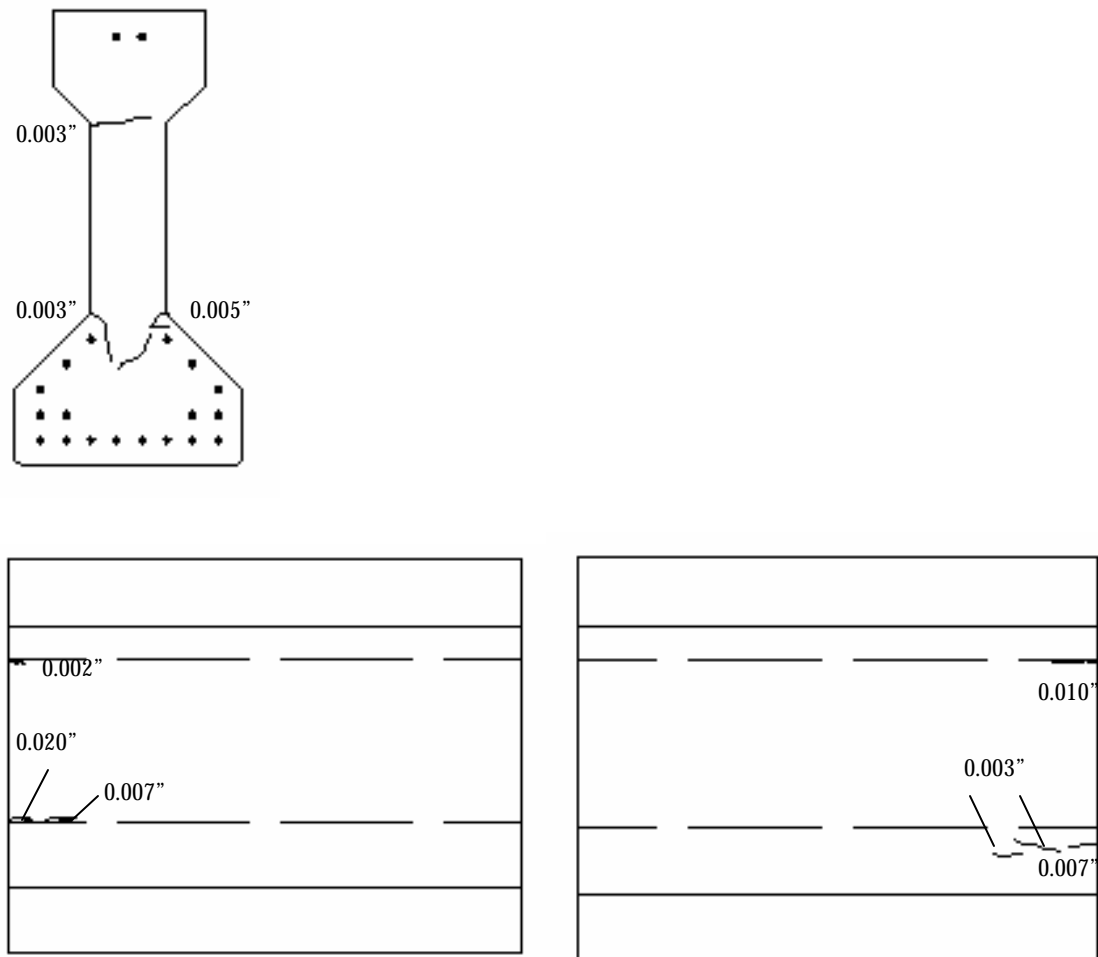


Figure 109. Beam End 3A: West End (top), Southwest (left), Northwest (right) (6 months)

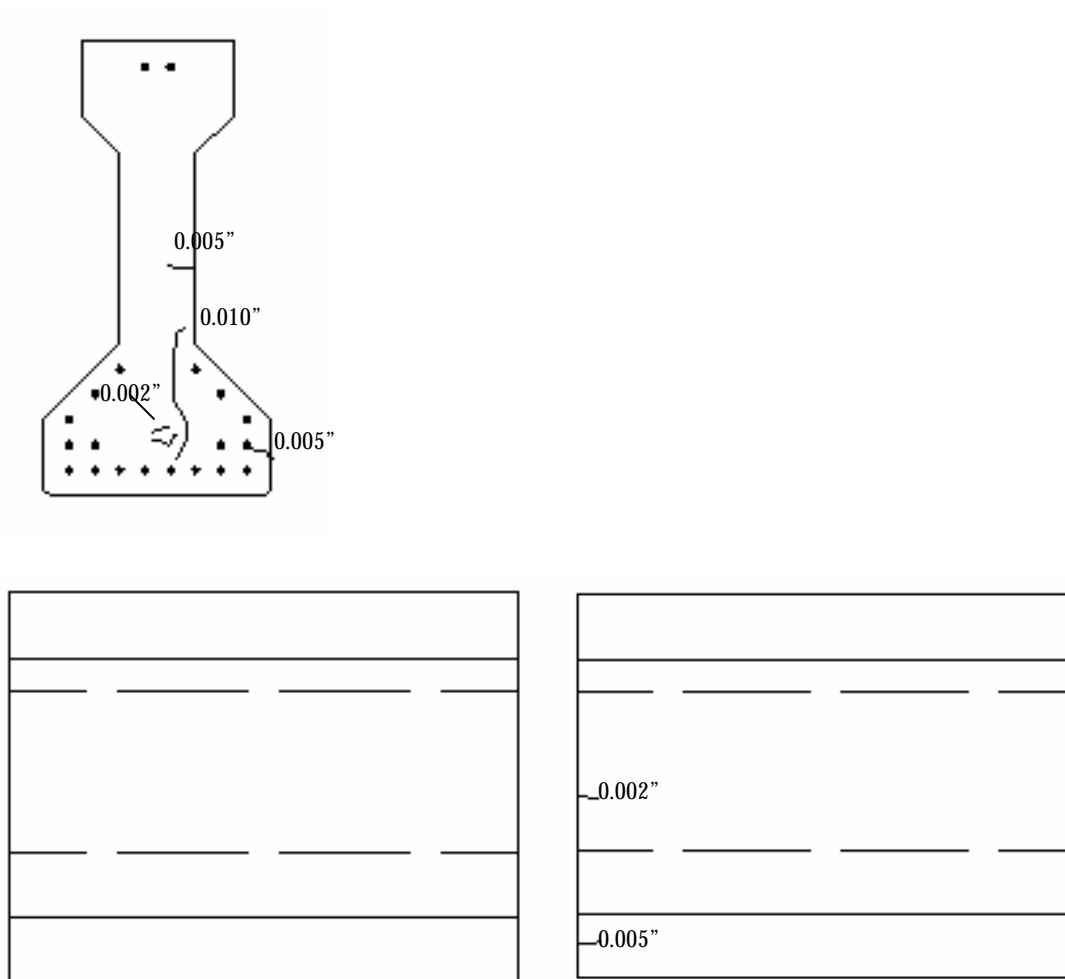


Figure 110. Beam End 3B: East End (top), Southeast (left), Northeast (right) (6 months)

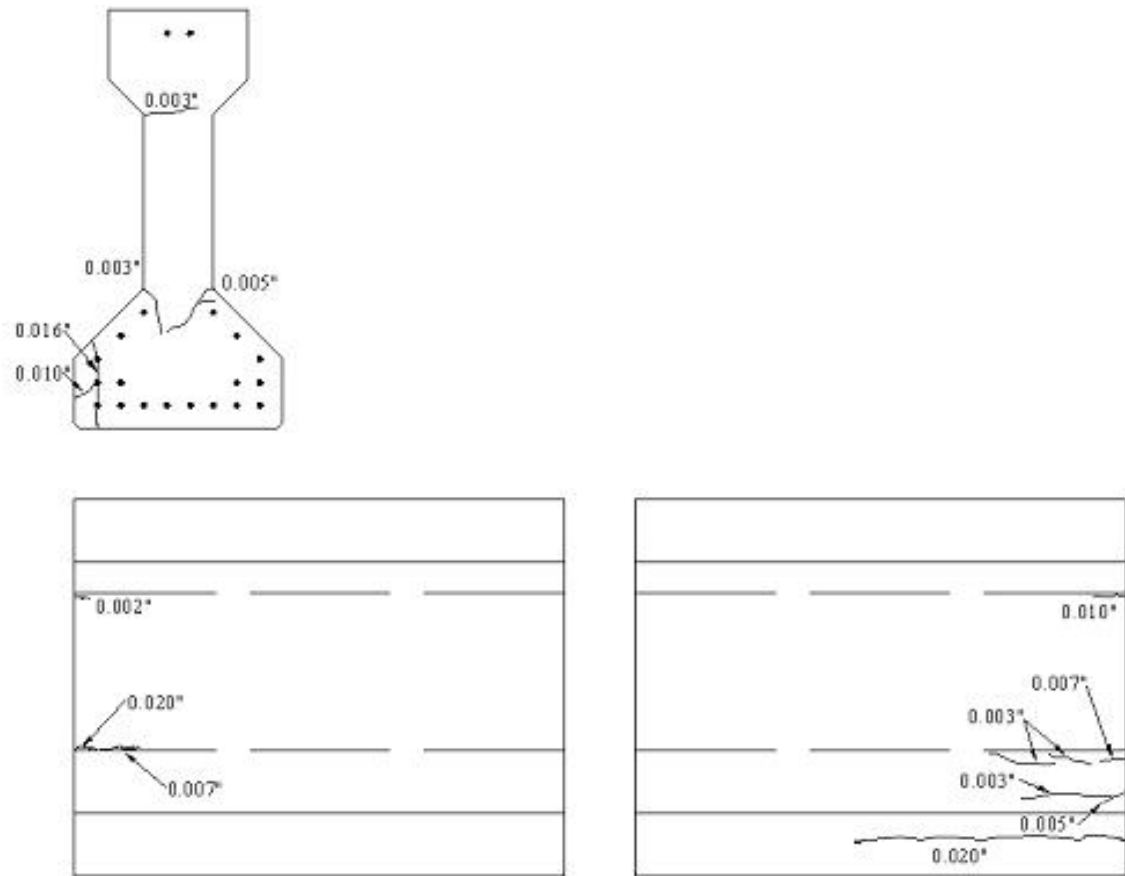


Figure 111. Beam End 3A: West End (top), Southwest (left), Northwest (right) (18 months)

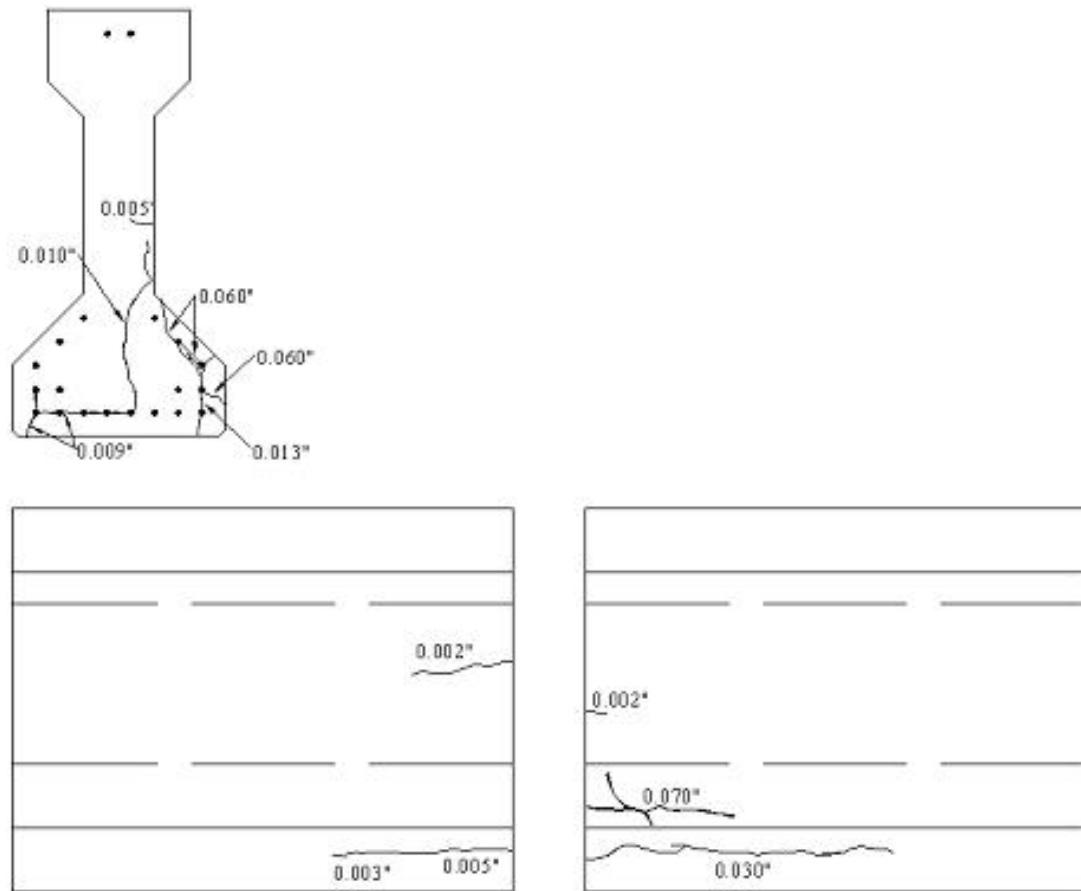


Figure 112. Beam End 3B: East End (top), Southeast (left), Northeast (right) (18 months)

5.3.4 Beam 4

Crack maps for both beam-ends were obtained at the completion of the first 6 months of exposure and at the conclusion of all tests. Figures 113 and 114 illustrate the crack maps for the west end 4A (untreated, post-polymer) and east end 4B (untreated, post-FRP). The crack widths on the west face varied from 0.003 to 0.009 inches. The northwest face crack width was 0.002 inches and the southwest face crack width was 0.005 inches. The crack widths on the east face ranged between 0.002 and 0.003 inches. The northeast crack width was 0.003 inches. No cracks were observed on the southeast face after 6 months of exposure.

Figures 115 and 116 show crack maps after 18 months of exposure for the 4A and 4B beam ends, respectively. Additional cracking developed at the end face of 4A from 6 months to 18 months.

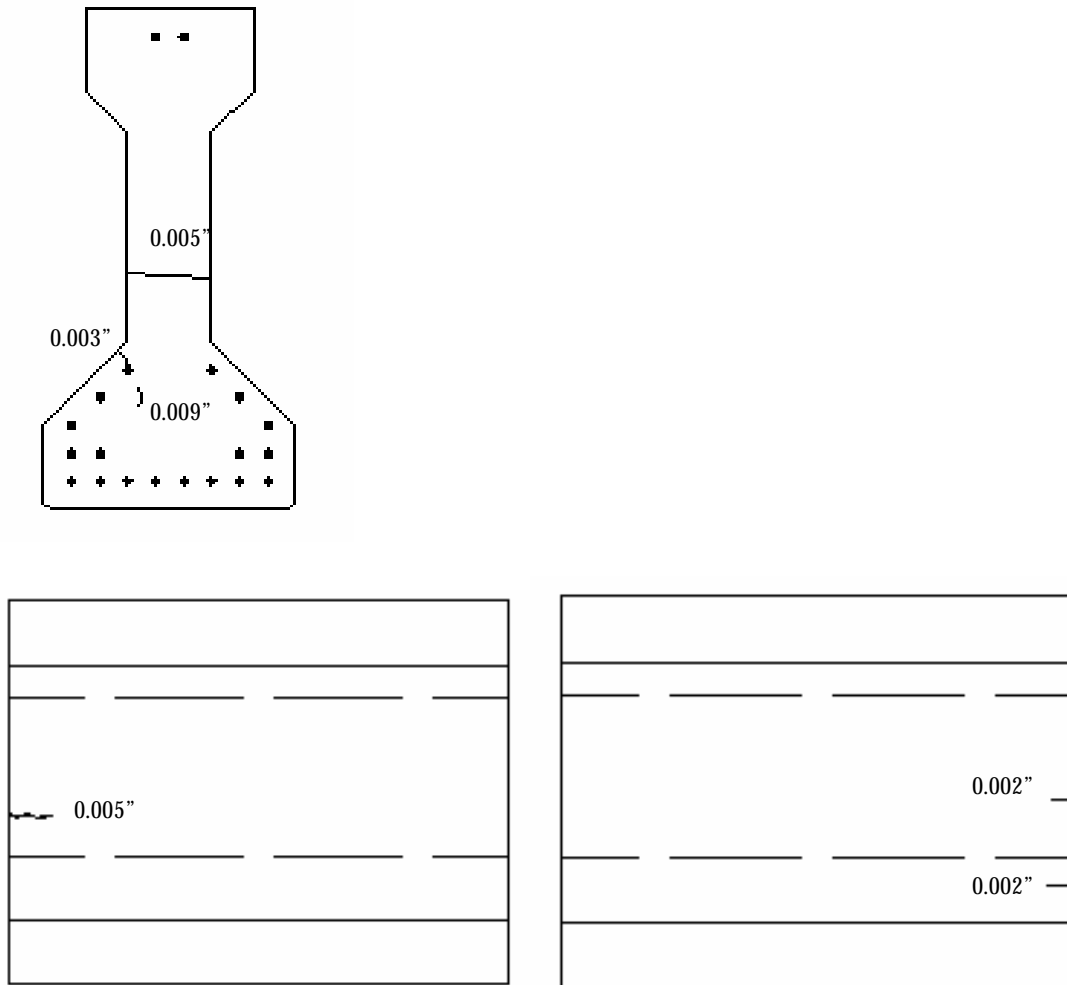


Figure 113. Beam End 4A: West (top), Southwest (left), Northwest (right) (6 months)

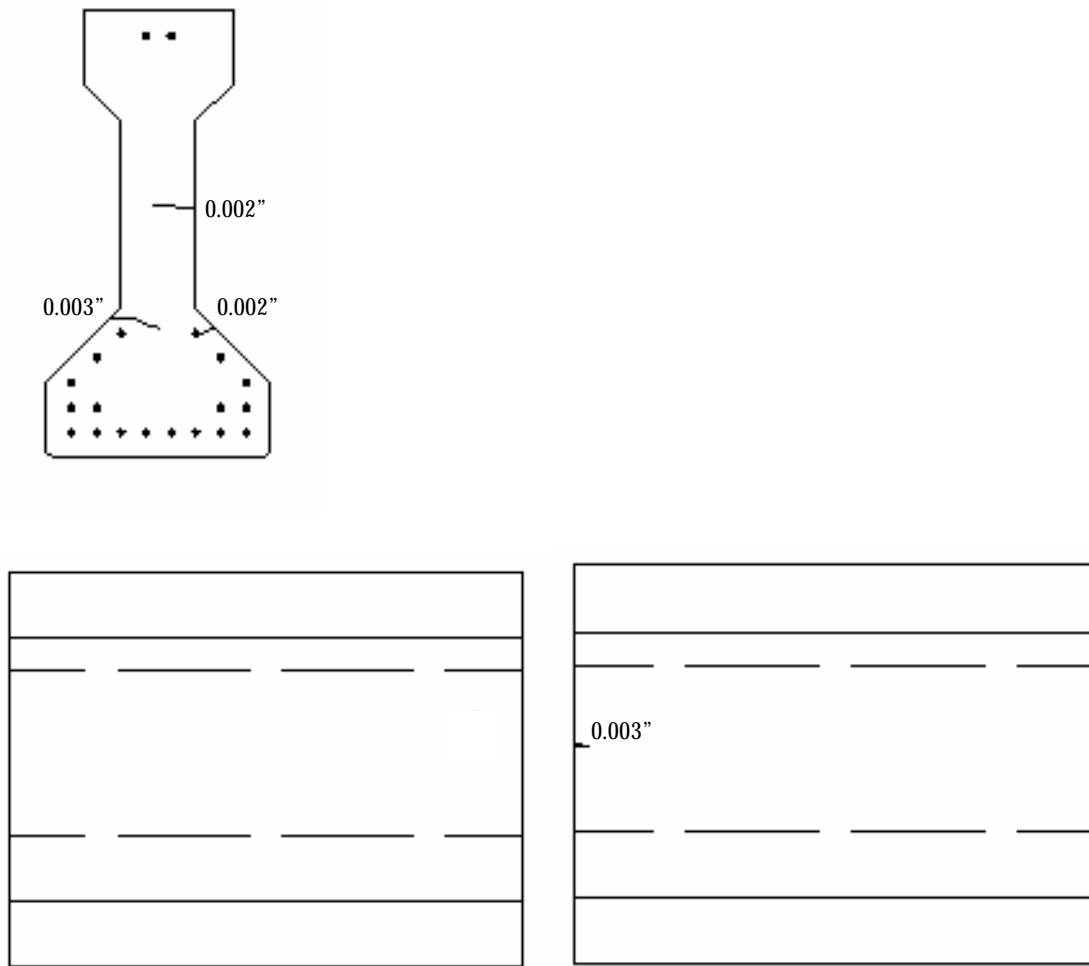


Figure 114. Beam End 4B: East (top), Southeast (left), Northeast (right) (6 months)

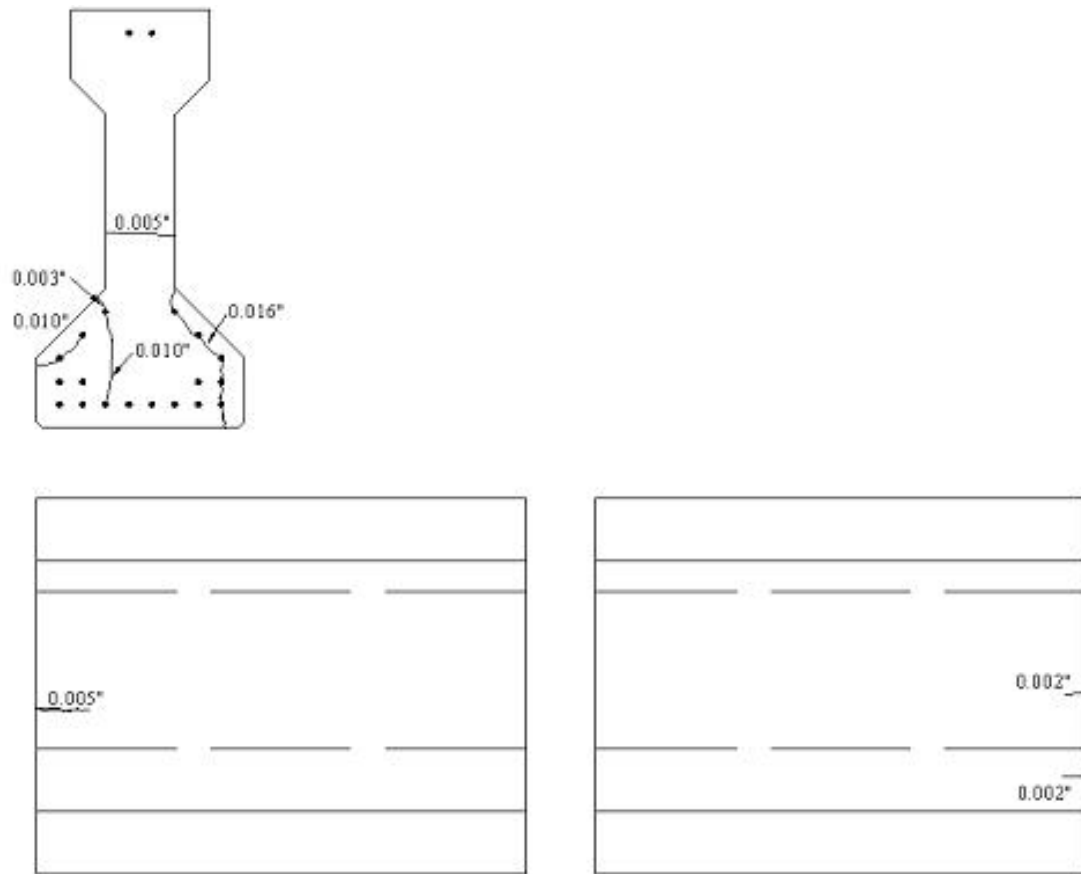


Figure 115. Beam End 4A: West (top), Southwest (left), Northwest (right) (18 months)

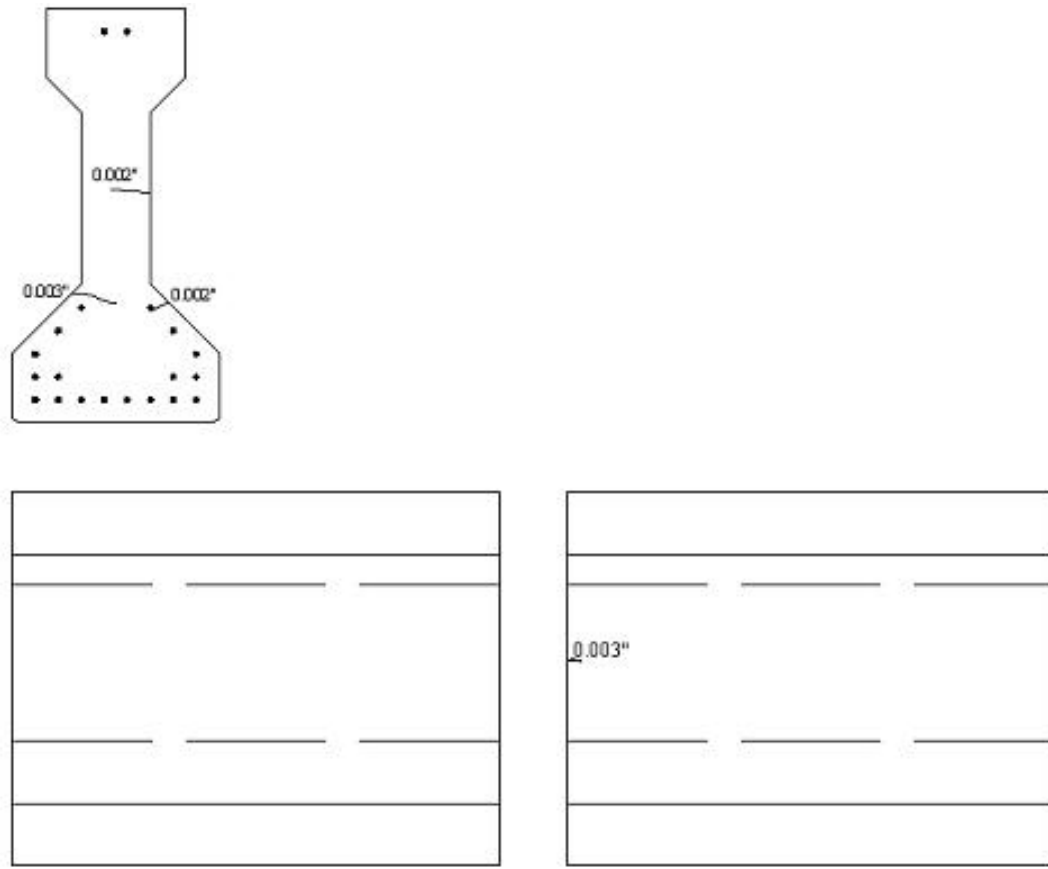


Figure 116. Beam End 4B: East (top), Southeast (left), Northeast (right) (18 months)

5.3.5 Beam 5

Both ends of beam 5 were coated with polymer resin (5A) or FRP wrap (5B) from the first day. Cracking was not observed in 5A, and was not detectable because of FRP wrap in 5B. Therefore crack mapping is not shown here.

5.3.6 Comparison of Crack Map Results

To compare various crack maps together, a rating scale of 1 to 8 was utilized with 1 representing least (almost no) cracking, and 8 representing the most extensive cracking observed. The range 1 to 8 was selected because the other comparative scale utilized for corrosion comparisons (the PCI reference method discussed in next section of this report) also uses a 1 to 8 numerical scale. Table 32 shows the rating numbers given to various beam-ends at the conclusion of all tests (after 18 months of exposure). The shaded rows in Table 32 indicate treatment after 6 months of exposure.

Table 32. Numerical Rating* of Extent of Cracking Observed After 18 Months of Exposure

BeamEnd	Description	Rating
1A	Epoxy Coated From Day 1	2
1B	Epoxy Coated After 6 Months of Exposure	4
2A	No Treatment Applied	6
2B	Patch Repair After 6 Months of Exposure	7
3A	Silane Sealer Applied from Day 1	5
3B	Silane Sealer Applied After 6 Months of Exposure	8
4A	Polymer Resin Coating Applied After 6 Months of Exposure	3
4B	FRP Wrap Applied After 6 Months of Exposure	1
5A	Polymer Resin Coating Applied From Day 1	1
5B	FRP Wrap Applied From Day 1	1

*Rating is based on 1 –8 scale, 1 indicating least cracking, 8 most extensive cracking
Shaded rows indicate beam-ends that were treated after 6 months of exposure

It is clear from Table 32 that among beam-ends treated from the first day of exposure, beam-ends 5B, 5A, 1A, 3A, and 2A were ranked from best to worst. The FRP wrap and polymer resin coatings applied from Day 1 provided better performance as far as extent of cracking is concerned. For beam-ends treated after 6 months of exposure, the least to most extensive cracks were observed in 4B, 4A, 1B, 2B and 3B. Again, FRP wraps and polymer resin coating performed the best in this case.

5.4 DISSECTION OF BEAM ENDS

At the conclusion of 18 months of exposure to accelerated corrosion environment, all beam-ends were partially dissected to closely examine the condition of strands in the bottom flanges. Concrete in the southern half of bottom flanges in all beam-ends was removed using a jackhammer. Two strands were removed from each dissected beam-end. These strands represented the worst condition, with respect to corrosion, observed along the sloping / vertical sides of the flange and along the bottom layer of strands in the exposed area. One strand was removed from the sloping area and the other from the exposed bottom layer of strands. The corrosion conditions of the removed strands were categorized based on a visual ranking proposed by Sason in a PCI Journal paper [55]. In the PCI method, the surface condition of strands is compared against a set of pictures of strands with various corrosion states. Based on this comparison, a numerical rating from 1 to 8 (1 best, 8 worst) is given to each strand sample removed.

Figures 117 thru 128 show the condition of strands after dissection for all beam-ends. A numerical rating of bottom strand samples based on the PCI reference [55] are given in Table 33. Among the beam-ends treated from the first day, the FRP wrap and polymer resin coating were rated the best. For beam-ends treated after 6 months of exposure, the Silane sealer and polymer resin coating were judged the best with respect to strand corrosion.

Table 33. Numerical Rating* of the Extent of Corrosion Observed on Strands After 18 Months of Exposure

Beam End	Description	Side Strand	Bottom Strand	Ave. Rating
1A	Epoxy Coated From Day 1	2	4	3
1B	Epoxy Coated After 6 Months of Exposure	7	7	7
2A	No Treatment Applied	4	7	5.5
2B	Patch Repair After 6 Months of Exposure	8	8	8
3A	Silane Sealer Applied from Day 1	2	5	3.5
3B	Silane Sealer Applied After 6 Months of Exposure	5	6	5.5
4A	Polymer Resin Coating Applied After 6 Months Exp.	6	6	6
4B	FRP Wrap Applied After 6 Months of Exposure	7	7	7
5A	Polymer Resin Coating Applied From Day 1	2	2	2
5B	FRP Wrap Applied From Day 1	1	3	2

*Rating is based on 1 –8 scale, 1 indicating least corrosion, 8 most extensive corrosion
Shaded rows indicate beam-ends that were treated after 6 months of exposure



Figure 117. Beam End 1A – Treated With Epoxy Coating From Day 1



Figure 118. Beam End 1B – Treated With Epoxy Coating After 6 Months of Exposure



Figure 119. Beam End 2A – Not Treated At All



Figure 120. Beam End 2B – Patch Repair After 6 Months of Exposure



Figure 121. Beam End 3A – Treated With Silane Sealer From Day 1



Figure 122. Beam End 3B – Treated With Silane Sealer After 6 Months of Exposure



Figure 123. Beam End 4A – Treated With Polymer Resin Coating After 6 Months of Exposure



Figure 124. Beam End 4B – Treated With FRP Wrap After 6 Months of Exposure



Figure 125. Beam End 5A – Treated With Polymer Resin Coating From Day 1



Figure 126. Beam End 5B – Treated With FRP Wrap From Day 1



Figure 127. Comparison of Strands with Respect to Corrosion



Figure 128. Comparison of Strands with Respect to Corrosion

5.5 ASSESSMENT OF RESULTS

To compare the performance of various beam-ends, it was decided to select three performance indicators: chloride intrusion, extent of cracking, and extent of corrosion. All these three parameters were previously given a numerical rating of 1 (best) to 8 (worst). The ratings given were based on visual observations, except for the chloride levels, which were based on actual chloride measurements. The corrosion ratings were based Sason's rating system published in PCI Journal [55]. Table 34 lists all numerical ratings (from Tables 30, 32, and 33) and the sum of the three ratings for each beam-end, suggesting that the three indicators were given equal weight in the final summation. The decision to give equal weight to the three indicators was a subjective yet rational choice based on the fact that corrosion of the strand could occur due to chlorides and moisture reaching strands from two sources: (1) diffusion through concrete surface, and (2) entry thru interstitial spaces in between wires at the cut end of strand.

It is clear that among all beam-ends including those that were treated from the first day, the polymer resin coatings and the FRP wraps provided the smallest overall rating number (i.e. the best overall condition). The patch repair had the largest overall rating number (23), which indicated that the repair was not effective in its intended function. Among the beam-ends that were treated after 6 months of exposure, the FRP wrap had the best overall rating, followed closely by the polymer resin coating and epoxy coating. It is clear that protecting the beam-ends from day 1 is by far the best long-term approach. The untreated beam-end (2A) showed performance comparable to those of treated beam-ends after 6 months of exposure (except the patch repairs). As explained earlier, this is likely due to a noted electrical problem in this

beam-end where loose connections may have moderated the effect of the accelerated corrosion process.

Considering that the cost and effort involved in installing FRP wraps, especially in existing structures, far exceed those of the polymer resin or epoxy coatings, it is recommended that polymer resin coatings or epoxy coatings be used instead.

Table 34. Comparison of various Beam-End Numerical Ratings and Overall Ratings*

Beam End	Description	Chlorides	Cracking	Corrosion	Overall Rating
1A	Epoxy Coated From Day 1	1	2	3	6
1B	Epoxy Coated After 6 Months of Exposure	2.5	4	7	13.5
2A	No Treatment Applied	2	6	5.5	13.5
2B	Patch Repair After 6 Months of Exposure	8	7	8	23
3A	Silane Sealer Applied from Day 1	1	5	3.5	9.5
3B	Silane Sealer Applied After 6 Months of Exposure	2	8	5.5	15.5
4A	Polymer Resin Coating Applied After 6 Months Exp.	4.5	3	6	13.5
4B	FRP Wrap Applied After 6 Months of Exposure	2.5	1	7	10.5
5A	Polymer Resin Coating Applied From Day 1	1	1	2	4
5B	FRP Wrap Applied From Day 1	1.5	1	2	4.5

*Individual criterion ratings were based on 1 –8 scale, 1 indicating best effect, 8 indicating worst effect. The overall ranking was based on a scale of 3 to 24 with 3 indicating the best condition and 24 indicating the worst condition.

Shaded rows indicate beam-ends that were treated after 6 months of exposure

PROPOSED FIELD EVALUATIONS

6.1 FIELD EVALUATION PLAN

It is suggested that the treatment methods that were determined to be effective in this laboratory study be also evaluated on actual bridges in the field as part of a future study. It is proposed that the Wisconsin Department of Transportation identify a total of 10 to 15 candidate bridges for field demonstrations. Ideal candidates would consist of at least two to four pairs of new prestressed girder bridges. Each pair should be similar (preferably identical) and constructed in relative proximity to each other or on the same highway. Planning and allowance should be made in the contract drawings for localized surface treatments of the beam-ends (the last 2 ft) in one bridge in each pair using polymer resin coating or epoxy coating. These treatments should be applied before placement on the pads in the field so that all exposed surfaces within the coverage area are coated. The untreated bridges would serve as control bridges. The aesthetic issues involved in applications of coatings should be addressed, especially when the girders would be visible to the public from under the bridge. As a minimum, the color of the coating should be as close as possible to the untreated concrete. It is also possible to apply the treatment in a slightly larger area to provide a decorative pattern (perhaps an arch pattern) on the two exterior girder faces.

It is proposed that measurements be taken at yearly intervals. Such monitoring would include half-cell potential measurements and visual condition surveys (cracking, spalling, etc.). Since half-cell potential measurements cannot be performed over the coatings, it is suggested that a

small, untreated opening (circular) be planned in the treatment area to allow half-cell measurements. The opening can have a cover to prevent chloride and moisture penetrations. However, steps must be taken to ensure that the opening does not affect or compromise the performance of coating.

In addition to the proposed treatments on new structures, it is also proposed that at least three pairs of existing bridges be identified as candidate bridges for evaluation of treatment applications on existing bridges. Each pair should be similar (preferably identical) and constructed in relative proximity to each other or on the same highway. It is recommended that these bridges be less than 15-20 years old to limit the pre-existing corrosion and chlorides in the beam-ends. The polymer resin coating or epoxy coating should be applied to the exposed areas of all beam-end in half of these bridges. Yearly half-cell measurements and surveys similar to those proposed for new bridges should be implemented. It is suggested that all these bridges be monitored for a time period of at least 5 to 10 years.

SUMMARY, CONCLUSIONS, AND RECOMMENDATIONS

7.1 SUMMARY AND CONCLUSIONS

The primary objectives of this research were: (1) to collect and synthesize information on repair and rehabilitation methods for concrete bridges (2) to evaluate the effectiveness of preventative and corrective methods to address deterioration of prestressed bridge beam-ends and (3) to initiate development of an expert system software program to assist in the assessment, diagnosis, and repair of concrete bridges.

A thorough understanding of the state-of-the-art in the field of rehabilitation of concrete bridges, especially in northern climates, was considered crucial for the success of this effort. Therefore, a comprehensive review of available literature in relevant subject areas was performed. On-line sources of information, as well as conventional search databases were utilized. An extensive literature database was developed using Microsoft Access. Over 570 papers were cataloged, and include such searchable information as the title, publisher, author, and date. The database also includes the abstracts or summaries of many of the papers. The user can search the database by performing a keyword, title, or author query.

The following general conclusions can be drawn regarding the repair methods for concrete bridges based on the results of the literature review. A detailed discussion on each subject is given in chapter 2. Surface treatments, while reasonably effective over the short-term, have demonstrated limited effectiveness over the long term, unless they are applied prior to chloride

contamination. Cathodic protection, while effective, is not commonly employed due to the high component and maintenance costs as well as the complexity of the method. In addition, due to the possibility of hydrogen embrittlement, cathodic protection of prestressed concrete beams is generally not recommended. Research studies have established the effectiveness of FRP composites to prevent and mitigate corrosion-damage in concrete columns.

An initial version of an expert system computer program, Concrete Bridge Assessment and Rehabilitation (ConBAR), was developed to assist in the diagnosis of concrete bridge deterioration problems and to identify repair, rehabilitation, or preventative maintenance options. This program includes a user-friendly interface that obtains relevant information on the subject bridge through a series of questions, and provides suggestions and recommendations to the user. The depth and variety of questions that ConBAR asks the user before making recommendations far exceed the scope of previous attempts at developing such expert system tools for concrete bridges. This necessitates a very large set of expert rules (based on combinations of possible answers) that must be incorporated into the program. This program currently includes the complete infrastructure required as well as a limited number of expert rules, which must be expanded and enhanced in future developments of this program. It is important to emphasize that the tools developed are intended and expected to assist and facilitate the work of experienced maintenance personnel, and not to replace it.

ConBAR addresses cracking, surface defects (such as honeycombing and blistering), spalling, corrosion, vehicle impact damage, alkali silica reactivity (ASR), and chemical exposure. The system also considers exposure conditions, previous repairs, bridge age, inspection information and other factors. The knowledge base includes: (1) facts and rules of thumb, (2) visual

information such as photographs and drawings, (3) access to a rehabilitation literature database and (3) descriptive statements. ConBAR provides a number of possible solutions along with their pros and cons, a suggestion, or a hypothesis. Recommendations for additional tests or sources of information are supplied to confirm or refute the hypothesis.

Based on the results of the literature review, a test plan and repair concept were submitted and approved by the Project Oversight Committee, appointed by the project's sponsor, the Wisconsin Department of Transportation (WisDOT). The work plan included performing laboratory tests on five new 8-foot long prestressed concrete bridge I-beams to address corrosion-damage and subsequent repair of beam ends due to chloride-laden water infiltrating through faulty expansion joints. The beam-ends were subjected to wet/dry cycles of salt laden water to accelerate the corrosion process. In addition to the salt-water exposure, the beam-ends were subjected to an impressed electric current to assist in accelerated corrosion. Two "cathode" bars were placed in the beam and the entire reinforcement system (strands and bars) was made anodic. This created a "reverse cathodic protection" system, thus accelerating the penetration of chloride ions and the initiation of steel corrosion. Some end regions were pretreated with a sealer, epoxy coating, polymer (resin), or fiber-reinforced polymer (FRP) composite wrap to assess their effectiveness in protecting the beam when subjected to an accelerated corrosive environment.

As was done initially, sealer, epoxy coating, polymer (resin), and FRP wrap treatments were also applied to other previously untreated beam-ends after an initial exposure period of 6 months. In addition, one beam-end was patch repaired with no additional protection system to compare its performance with other systems. After the repairs were completed and the

surface treatments applied, the beam-ends were again subjected to an accelerated corrosion regime. The overall exposure was extended from a period of 12 months to 18 months due to limited progress in initiating widespread corrosion in the beam-ends. A number of test parameters were measured during the monitoring period. The total accelerated corrosion exposure period for all specimens was 18 months.

The chloride content of the beams was determined prior to exposing the specimens to the accelerated corrosion environment, after the completion of 6 months of exposure, and after 18 months of exposure. The initial chloride content was measured in the middle area of one beam specimen. The chloride content was determined to be higher than expected. The mix water and aggregate sources used were then tested to identify the source of chlorides. It was determined that aggregates were the likely source. However, it is not clear, with the current level of testing, whether or not these chlorides are permanently bound within the aggregates. Chloride samples following the first 6 months of the accelerated corrosion regime were also taken at two locations on the bottom flange of the beam-end receiving the patch repair. The measurements indicated relatively high chloride concentrations near the surface, with the values decreasing with increasing distance from the surface. This is consistent with the behavior of chloride ions migrating into the concrete. The corrosion threshold was exceeded at all depths measured (up to 1 inch from the concrete surface) at a distance of 2 inches from the end of the beam. At the location of 6 inches from the face of the beam, the corrosion threshold level was exceeded up to a depth of $\frac{1}{2}$ inch.

Chloride samples were also taken at all beam-ends at the conclusion of the entire 1 $\frac{1}{2}$ -year exposure prior to dissection. These measurements were made on samples taken in the middle

of the sloping surface of the bottom flange at a distance of 2 inches from the back of the beam. The highest chloride levels were observed in the beam-end with patch repairs. It appears that the interface between the old and new concretes may have allowed accelerated intrusion of chlorides deep into the patch and old concrete. The chloride contents for the beam-ends that were treated with epoxy coating, polymer resin coating or FRP from the first day clearly show significantly lower chloride contents than other specimens. The beam-end treated with Silane sealer from the first day had far less chlorides than the untreated beams (or beams treated after 6 months). However, the chloride levels for this beam-end were higher than the corresponding beams treated with epoxy coating, polymer resin coating or FRP from Day 1. The specimens that were treated after 6 months show high levels of chlorides, but they are less than the beam with patch repair. Among the beam-ends that were treated after 6 months of exposure, the Silane sealer and the epoxy coatings had the least chloride contents.

A regulated voltage of 9V was applied continuously over the course of the exposure cycles to facilitate the corrosion process and speed the intrusion of chlorides. The corrosion currents versus time data for each of the beams were collected and recorded with a data acquisition system. All corrosion currents exhibit a decrease in value with time. This reduction is commonly observed in such experiments, and is due in part to the fact that corrosion products increase the resistance at the surface of strands. The cumulative area under the corrosion current versus time graph is indicative of the amount of steel loss due to corrosion. However, after careful examination of all results, it became clear that corrosion rate measurements were not an effective method for overall corrosion assessments in the localized beam-end areas. This may have been due to the fact that the measured corrosion rates were an indication of

overall corrosion in the overall reinforcement system rather than the localized effect in the beam-ends.

In addition, periodically, half-cell potential readings were obtained. The half-cell measurements were only taken on the non-treated beam-ends since surface treatments provide a non-conductive barrier that renders the half-cell measurements ineffective. Expansion measurements were also periodically taken at each beam-end. However, due to problems encountered with the metal points corroding, the accuracy of the measuring device, and issues with keeping the points attached to the concrete surface, it was determined that the readings were inconsistent and not representative of accurate strain measurements. The specimens were also visually monitored for cracking and spalling. Detailed crack maps were sketched at the end of the first 6-month corrosion exposure cycle, and at the conclusion of all tests.

Half-cell measurements using a copper-copper sulfate electrode were obtained for each beam end. Initial half-cell potentials were relatively uniform at all points measured. Whereas the half-cell readings after the first exposure cycle varied, depending on their location on the beam. As expected, the values increased substantially as measurements neared the end of the beam. The highest readings were located on the bottom flange near the edge of the beam. These readings were consistent with the flow of the salt water down the end of the beam, which normally coincides with corrosion regions.

After six months of exposure to a corrosive environment, the half-cell readings for the untreated beam-end (opposite of the initially epoxy coated end) indicated that corrosion was not occurring. The half-cell readings for the four ends of the two initially untreated beams indicated that it was inconclusive whether or not corrosion was occurring. However, the

untreated beam-end (opposite of the initially sealed end) possessed some half-cell readings on the bottom flange outer corner that indicated that there was 90% likelihood that corrosion was occurring at these regions.

After 10 months of exposure, the patched beam-end yielded a higher potential in comparison to the untreated end. The half-cell readings for the patched end were large enough to fall into the category that, with 90% probability, corrosion was likely occurring. Whereas the readings for the untreated end indicated that it could not be determined whether or not corrosion was occurring. It was concluded that, the higher potential indicated that the likelihood of corrosion occurring in the patched end is greater than in the untreated end. The half-cell potential readings for beam ends 2A (no treatment) and 2B (patched) after 18 months of exposure clearly showed corrosion activity in the beam-ends.

At the completion of the first exposure cycle all beam-ends had heavy salt residue along the front faces and on some portions of the bottom flanges. Rust stains were also evident along the path of the salt water. No major spalling or cracking was observed at that time. Some flaking of concrete was observed at the corners of the beam. In addition, corrosion products were observed on the exposed tendon ends, and were found to increase in amount over the course of the exposure. Since the first 6-month exposure cycle did not result in the concrete spalling or significant tendon corrosion, the configuration of the saltwater dispersion system was altered slightly to increase the likelihood of corrosion. The altered salt-water distribution setup was able to disperse water to both the sides and face of the beams. An 18-inch long concrete region of one of the untreated beam-ends was removed with a chipping hammer to facilitate installation of the patch repair. Corrosion products were observed mainly at the end

regions of the tendons. The build-up of corrosion products was seen to decrease as the distance from the edge of the end increased.

At the end of the 18-month exposure period, the beams were experiencing significant rust staining and salt residue deposits. Cracking was evident in many beam-ends. All beam-ends were crack-mapped and subsequently partially dissected. The state of corrosion of strands in each beam-end was numerically classified. The final decision on effectiveness of various methods was based on numerical ratings given to three indicators: chloride content, extent of cracking, and extent of observed strand corrosion. Each of the three parameters was given equal weight in determining the overall ratings. The following table illustrates the final ratings given.

Table 34. Comparison of various Beam-End Numerical Ratings and Overall Ratings*

Beam End	Description	Chlorides	Cracking	Corrosion	Overall Rating
1A	Epoxy Coated From Day 1	1	2	3	6
1B	Epoxy Coated After 6 Months of Exposure	2.5	4	7	13.5
2A	No Treatment Applied	2	6	5.5	13.5
2B	Patch Repair After 6 Months of Exposure	8	7	8	23
3A	Silane Sealer Applied from Day 1	1	5	3.5	9.5
3B	Silane Sealer Applied After 6 Months of Exposure	2	8	5.5	15.5
4A	Polymer Resin Coating Applied After 6 Months Exp.	4.5	3	6	13.5
4B	FRP Wrap Applied After 6 Months of Exposure	2.5	1	7	10.5
5A	Polymer Resin Coating Applied From Day 1	1	1	2	4
5B	FRP Wrap Applied From Day 1	1.5	1	2	4.5

*Individual criterion ratings were based on 1 –8 scale, 1 indicating best effect, 8 indicating worst effect. The overall ranking was based on a scale of 3 to 24 with 3 indicating the best condition and 24 indicating the worst condition.

Shaded rows indicate beam-ends that were treated after 6 months of exposure

7.2 RECOMMEDATIONS

Based on the results of this research effort, the following recommendations are made:

- The most effective solution for protection of prestressed concrete beam-ends is determined to be treating the beam-ends from the first day, i.e. before installation in the field. The treatment area would be limited to all surfaces within a 2-ft-length at the two ends of each beam. This includes the back end surface and the bottom surface. When the strands are cut flush with the back of the beam, the treatment must cover the cut end well to prevent horizontal migration of chlorides through interstitial spaces between wires. In cases where the strands are not cut flush (i.e. embedded in the diaphragm concrete), the exposed strand must be coated well to prevent horizontal chloride migration. This approach (treatment from the first day) is far more effective, and easier, than subsequent treatments in the field. The carbon fiber-reinforced polymer (FRP) coating, and polymer resin coating (FRP without fiber) were found to be the most effective treatments. Epoxy coating was the next best solution followed by silane treatment. As expected, leaving the beam-end untreated resulted in the worst overall performance compared to beam-ends that were treated from day 1.
- Considering that the FRP wrap, polymer resin coating, and epoxy coating were generally effective, it is recommended that either polymer (resin) coating or epoxy coating be used in new construction to protect the prestressed concrete beam-ends. The FRP wraps did not significantly improve performance over polymer resin coating, and would only add to the cost and difficulty of treatment. Since protecting the end face of the beam and the cut ends of the strands are crucial, it is recommended that

such treatments be performed in advance of installation in the field. The presence of diaphragms, bearings or other obstructions would likely make the field application of coatings to the beam-ends very difficult; especially after the diaphragm and deck concrete is cast.

- For existing prestressed concrete beam-ends, it is recommended that the protective treatments be applied as soon as possible, before chloride levels increase significantly. It is expected that the applications of polymer resin coating or epoxy-coatings to the exposed surfaces of the beam-ends in the field would contribute, albeit not as effectively, to the protection of beam-ends in the long run, if such treatments are implemented before chloride contaminations and corrosion have taken hold. In such cases, all exposed surfaces should be treated with either polymer resin coating or epoxy coating. The extent of pre-existing chloride contamination can be measured in the field (on the bottom flange at about 2 inches from the end of the beam) and compared with chloride contents measured in areas not exposed to chloride contaminations.
- In cases where corrosion and damage is advanced and has resulted in cracking and spalling of the beam-ends, the conventional patching alone would likely not be a durable repair method. Although not tested in this experimental effort, a patch repair that is subsequently coated with polymer resin coating or epoxy coating would likely provide a more effective repair.
- Although the above results and recommendations were based on tests on beam-ends, it is expected that they would also be applicable to pier elements (such as pier caps and columns) and abutments.

- The development of the ConBAR expert system program should be continued to include additional decision rules, deterioration cases and repair methodologies.
- The developed repair database should be continuously augmented and perhaps incorporated into ConBAR.
- The proposed field investigation detailed in Chapter 6 of this report should be considered for implementation.

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APPENDIX A

ConBAR Examples

Example 1: Extensive Damage

Member Observations:

The member under consideration is a beam located on a 25-year-old prestressed concrete girder bridge in Wisconsin. The beam end zones are exhibiting signs of distress. This 2-span bridge is located in a metropolitan area with an ADT of 18,500. The bridge carries a state highway and spans over another state highway. It is not located near industrial sites with harmful emissions, but does carry heavy trucks. There are no weight limits posted and the bridge has not been classified as structurally deficient or functionally obsolete. Inspectors have given a rating of five to the beams. No vehicle impact damage has occurred and no previous repairs have been performed on the beam. The bridge does have leaky expansion joints, and no support settlement is observed.

Cracking is observed in the beam end zones, generally around spalled and spalling areas with crack lengths less than 12 inches. The cracks are not related to extraordinary loading and not related to flexural or shear loading. It is unknown if the crack planes run through the aggregates and no residue is observed around the cracks. The cracks have not been observed to move noticeably with temperature changes.

No other surface defects such as honeycombing, blistering, abrasion, scaling, or popouts are observed. Delaminations and spalling on about 15% of the affected member zone (beam ends) are observed. The bridge deck has been overlaid with an asphaltic overlay.

Although testing has not been performed, alkali silica reactivity (ASR) is not suspected. The sulfate content is low. The depth of the carbonation front is unknown. The member is exposed to deicing salts through leaky expansion joints. A testing laboratory has measured chloride contents in the affected zone. The water-soluble chloride contents at the depth of cover and half the depth of cover are 0.16% and 0.35% by weight of cement, respectively. The permeability of the concrete is not measured and is therefore unknown. Corrosion stains are observed on the concrete surface in the beam-ends, and moderate rust is observed on the exposed steel. The steel is not epoxy coated. The actual compressive strength of concrete is unknown since coring for tests have not been performed. The overall concrete quality is judged to be average.

A printout of the ConBAR session is presented on the following pages.

ConBAR

Concrete Bridge Assessment and Rehabilitation Expert Program

[Click here to begin](#)

This program is designed to assist engineers in assessment, diagnosis and repair of concrete bridges.

For questions or updates to this program please contact Professor Habib Tabatabai, Department of Civil Engineering and Mechanics, University of Wisconsin-Milwaukee.

(414)229-5166

3200 N. Cramer Street, Milwaukee, WI 53211

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CONTINUE

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Bridge Member

What part of the structure is under investigation?

Substructure	Superstructure
<input type="radio"/> Pier column	<input type="radio"/> Bridge deck slab
<input type="radio"/> Pier cap	<input type="radio"/> Railings/Barriers
<input type="radio"/> Pier wall	<input checked="" type="radio"/> Beams/Girders

beams/girders are under investigation

Age

What is the approximate age of the bridge?

☐ 0 to 5 years

☒ 5 to 25 years

☐ 25 to 50 years

☐ 50 years +

the member is 5 to 25 years old

Spans

What are the number of spans of the bridge?

☐ 1

☒ 2

☐ 3

☐ 4 or more

the bridge has 2 spans

ENTER DATA

Data Entry

Is the data entered correctly?

☐ Yes - accept data

☐ No - retry and correct

CONTINUE

Population

Is the bridge located in:

☒ Metropolitan environment (population greater than 200,000)
☐ Urban environment (population between 20,000 and 200,000)
☐ Rural Environment (population less than 20,000)

the bridge is located in a metropolitan area

Industrial Proximity

Is the bridge located down-wind from an industrial area with significant harmful (e.g. acidic) emissions?

☐ Yes
☒ No
☐ Unknown

the bridge is not located near harmful emissions

Environment

In what type of climatic environment is the bridge located?

☒ Northern deicing environment (e.g. Wisconsin)
☐ Marine (coastal) environment in moderate climate (e.g. Oregon Coast)
☐ Marine (coastal) environment in hot climate (e.g. Florida Coast)
☐ None of the above

☐ Moderate (non-coastal) environment (Tennessee)
☐ Hot and humid (non-coastal) environment (Georgia interior)
☐ Hot and dry (non-coastal) environment (Arizona)

the bridge is located in a northern deicing environment

ENTER DATA

Data Entry

Is the data entered correctly?

☐ Yes - accept data
☐ No - retry and correct

CONTINUE

Bridge Type

This bridge is located on:

☐ An interstate highway

☒ A state highway

☐ A county road

☐ A city street

the bridge carries a state highway

Spans Over

This bridge spans over:

☐ A creek

☐ A river

☐ A lake

☐ A city street

☐ A county road

☒ A state highway

☐ An interstate highway

the bridge spans over a state highway

ENTER DATA

Data Entry

Is the data entered correctly?

☐ Yes - accept data

☐ No - retry and correct

CONTINUE

ADT

What is the average daily traffic (ADT) present on the bridge?

☐ ADT < or = 1000

☐ 1000 < ADT < or = 10,000

☒ ADT > 10,000

the ADT is greater than 10,000

Truck Traffic

Does this bridge carry a large number of heavily loaded trucks?

☒ Yes

☐ No

☐ Unknown

the bridge carries heavily loaded trucks

ENTER DATA

Data Entry

Is the data entered correctly?

☐ Yes - accept data

☐ No - retry and correct

CONTINUE

Weight Limits Is this bridge posted for weight limits? <input type="radio"/> Yes <input checked="" type="radio"/> No <input type="radio"/> Unknown		the bridge is not posted for weight limits
Structurally Deficient Is this bridge officially classified as structurally deficient? <input type="radio"/> Yes <input checked="" type="radio"/> No <input type="radio"/> Unknown		According to the National Bridge Inventory, structurally deficient refers to inadequate structural sufficiency or waterway adequacy. The Condition Ratings for the deck, superstructure and substructure is considered to be in "poor condition". The Structural Evaluation is considered "intolerable" and the waterway and Waterway Adequacy is considered intolerable, requiring a high priority for replacement. the bridge is not classified as structurally deficient
Functionally Obsolete Is this bridge officially classified as functionally obsolete? <input type="radio"/> Yes <input checked="" type="radio"/> No <input type="radio"/> Unknown		According to the National Bridge Inventory, functionally obsolete is related to insufficient geometric capability of the bridge to carry traffic, including inadequate deck geometry, underclearance, or approach roadway alignment. Bridges which qualify as Structurally Deficient and Functionally Obsolete are excluded from the Functionally Obsolete category. the bridge is not classified as functionally obsolete
<div>ENTER DATA</div>		
Data Entry Is the data entered correctly? <input type="radio"/> Yes - accept data <input type="radio"/> No - retry and correct		
<div>CONTINUE</div>		

Bridge Rating

What is the NBI inspection rating for the bridge component under consideration (0-failure, 9-new)?

☐ 0

☐ 1

☐ 2

☐ 3

☐ 4

☒ 5

☐ 6

☐ 7

☐ 8

☐ 9

☐ Unknown

the bridge rating is 5

ENTER DATA

Data Entry

Is the data entered correctly?

☐ Yes - accept data

☐ No - retry and correct

CONTINUE

Vehicle Damage

Has the member been damaged by a vehicle?

☐ Yes

☒ No

the member has not been damaged by a vehicle

ENTER DATA

Data Entry

Is the data entered correctly?

☐ Yes - accept data

☐ No - retry and correct

CONTINUE

Drainage

Are there any drainage issues that negatively affect the bridge component under consideration?

☒ Yes (such as a leaky expansion joint or plugged drainage system)
☐ No
☐ Unknown there are drainage problems present

Settlement

Are there any indications or evidence of bridge support settlement?

☐ Yes (e. x. deck profile has unusual dips or rises)
☒ No
☐ Unknown there is no evidence of support settlement

ENTER DATA

Data Entry

Is the data entered correctly?

☐ Yes - accept data
☐ No - retry and correct

CONTINUE

Repair Type

Please indicate if a repair has been made and the type of repair performed:

No repairs have been made

no repairs have been made

ENTER DATA

Data Entry

Is the data entered correctly?

☐ Yes - accept data
☐ No - retry and correct

CONTINUE

Cracking

How would you characterize the extent of cracking, if any, in the member under investigation?

☒ Extensive
☐ Moderate
☐ Light
☐ Non-existent
☐ Unknown

the member is extensively cracked

ENTER DATA

Data Entry

Is the data entered correctly?

☐ Yes - accept data
☐ No - retry and correct

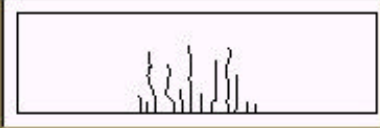
CONTINUE

Flexural Cracks

Is the cracking observed structural/flexural cracks?

☐ Yes
☒ No
☐ Unknown

the cracking is not structural/flexural




Shear Cracks

Is the cracking observed structural/shear cracks?

☐ Yes
☒ No
☐ Unknown

the cracking is not structural/shear



ENTER DATA

Data Entry


Is the data entered correctly?


☐ Yes - accept data
☐ No - retry and correct


CONTINUE


Crack Type


Please choose the type of crack observed.


☐ Craze cracks: fine random surface cracks or fissures



☐ Checking: development of shallow surface cracks at closely spaced but irregular intervals



☐ D-cracking: a series of cracks in concrete near and roughly parallel to joints, edges, and structural cracks


☐ Pattern cracking: fine openings on concrete surfaces in the form of a pattern; resulting from a decrease in volume of the material near the surface, or increase in volume of the material below the surface or both


☒ Transverse cracks: cracks that develop at right angles to the long direction of the member


☐ Diagonal cracks: In a flexural member, an inclined crack caused by shear stress, usually at about 45 degrees to the axis; or a crack in a slab, not parallel to either the lateral or longitudinal directions


☐ Plastic cracks: cracking that occurs in the surface of fresh concrete soon after it is placed and while it is still plastic


☐ Shrinkage cracks: cracking of a structure or member due to tensile failure in tension caused by external or internal restraints as reduction in moisture content develops, or as carbonation occurs, or both


☐ Temperature cracks: cracking due to tensile failure, caused by temperature gradient in members subjected to external restraints or by temperature differential in members subjected to internal restraints

Descriptions and photographs
 courtesy of ACI 201.1 R-92

Data Entry

Is the data entered correctly?

☐ Yes - accept data

☐ No - retry and correct

First Crack Observation

When was the cracking first observed?

☐ When the concrete was in plastic state
☐ Within the first few days after casting
☐ Within the first year after casting
☒ Several years after casting
☐ Unknown

the cracking was first noticed several years after casting

Crack Planes

Do crack planes go through the aggregates?

☐ Yes
☐ No
☒ Unknown

You can determine this by coring the concrete directly over the crack and investigating the crack and crack surface, or by looking at a spalled piece.

it is unknown if the crack planes transverse through the aggregates

Crack Residue

Are there any residues evident around some of the cracks?

☐ Yes, white residue (efflorescence)
☐ Yes, rust staining (corrosion)
☒ No
☐ Unknown

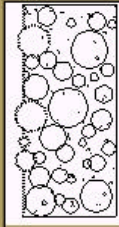
there is no residue around the cracks

Data Entry

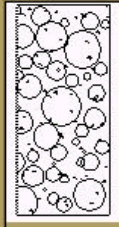
Is the data entered correctly?

☐ Yes - accept data
☐ No - retry and correct

CONTINUE



Crack planes do not go through aggregates



Cracks go through aggregates

ENTER DATA

Crack Orientation

The orientation of cracks can best be described as:

- ☒ Random short (<12") cracks
- ☐ Random long (>12") cracks
- ☐ Primarily longitudinal with cracks less than 6" apart
- ☐ Primarily longitudinal with cracks 6" to 18" apart
- ☐ Primarily longitudinal with cracks over 18" apart
- ☐ Primarily transverse with cracks less than 6" apart
- ☐ Primarily transverse with cracks 6" to 18" apart
- ☐ Primarily transverse with cracks over 18" apart
- ☐ Primarily inclined with cracks less than 6" apart
- ☐ Primarily inclined with cracks 6" to 18" apart
- ☐ Primarily inclined with cracks 18" apart
- ☐ Both longitudinal and transverse (perpendicular)
- ☐ Unknown

the cracks are random and short

ENTER DATA

Data Entry

Is the data entered correctly?

- ☐ Yes - accept data
- ☐ No - retry and correct

CONTINUE

Loading

Is the cracking related to extraordinary loading?

☐ Yes
☒ No
☐ Unknown

the cracking is not related to extraordinary loading

Crack Movement

Do crack widths change as temperature changes occur?

☐ Yes
☒ No
☐ Unknown

A crack gage can be placed across the crack to measure movements.

the crack widths do not change with temperature changes

ENTER DATA

Data Entry

Is the data entered correctly?

☐ Yes - accept data
☐ No - retry and correct

CONTINUE

Blistering

Is concrete blistering evident?

☐ Yes
☒ No

Blistering: the irregular raising of a thin layer, frequently 25 to 300 mm in diameter, at the surface of placed mortar or concrete during or soon after completion of the finishing operation; blistering is usually attributed to early closing of the surface and may be aggravated by cool temperatures.


concrete blistering is not evident

Honeycomb

Is honeycombing evident?

☐ Yes
☒ No

Honeycomb: voids left in concrete due to failure of the mortar to effectively fill the spaces among course aggregate particles.




concrete honeycombing is not evident

Abrasion

Is abrasion damage evident?

☐ Yes
☒ No

Abrasion Damage: wearing away of a surface by rubbing and friction



concrete abrasion damage is not evident

ENTER DATA


Descriptions and photographs courtesy of ACI 201.1 R-92

Data Entry

Is the data entered correctly?

☐ Yes - accept data
☐ No - retry and correct

CONTINUE

Delamination	
<p>Is delamination evident?</p> <p><input checked="" type="radio"/> Yes</p> <p><input type="radio"/> No</p> <p><input type="radio"/> Unknown</p>	<p>Delamination: a separation along a plane parallel to a surface generally near the upper surface; found frequently in bridge decks and other types of elevated reinforced concrete slabs and may be caused by corrosion of reinforcing steel; delaminations affects large areas and can often be detected by hammer sounding</p>  <p>Delamination can be determined using the following methods: chain drag, ground penetrating radar, and thermographic techniques</p> <p>Descriptions and photographs courtesy of ACI 201.1 R-92</p>
<p>concrete delamination is evident</p>	
<p>Delamination Size</p> <p>Approximately what percentage of the area of the element under investigation is delaminated?</p> <p><input type="radio"/> Less than 2%</p> <p><input type="radio"/> Between 2% and 5%</p> <p><input type="radio"/> Between 5% and 10%</p> <p><input checked="" type="radio"/> Between 10% and 15%</p> <p><input type="radio"/> Over 15%</p> <p><input type="radio"/> Unknown</p> <p><input type="radio"/> None of these apply</p>	
<p>concrete delamination is between 10% and 15%</p>	
<p>ENTER DATA:</p>	
<p>Data Entry</p> <p>Is the data entered correctly?</p> <p><input type="radio"/> Yes - accept data</p> <p><input type="radio"/> No - retry and correct</p>	
<p>CONTINUE</p>	

Overlay Type

What type of overlay, if any, has been applied to the bridge deck?

☐ Latex-modified concrete

☐ Portland cement concrete

☐ Low-slump dense concrete

☐ Silica fume concrete

☒ Asphaltic

☐ Other

☐ Unknown

☐ Not applicable

the overlay is asphaltic

ENTER DATA

Data Entry

Is the data entered correctly?

☐ Yes - accept data

☐ No - retry and correct

CONTINUE

Scaling

Is concrete scaling evident?

☐ Yes
☒ No


Scaling: local flaking or peeling away of the near-surface portion of hardened concrete or mortar; also of a layer from metal

concrete scaling is not observed


Type of Scaling

What is the extent of the scaling?


☐ Light: loss of surface mortar without exposure of coarse aggregate




☐ Medium: loss of surface mortar 5 to 10 mm in depth and exposure of coarse aggregate



☐ Severe: loss of surface mortar 5 to 10 mm in depth with some loss of mortar surrounding aggregate particles 10 to 20 mm in depth



☐ Very Severe: loss of coarse aggregate particles as well as mortar, generally to a depth greater than 20 mm



☒ None of these apply

Descriptions and photographs courtesy of ACI 201.1 R-92

none of these apply

ENTER DATA

Data Entry

Is the data entered correctly?

☐ Yes - accept data
☐ No - retry and correct

CONTINUE

Popouts

Are concrete popouts evident?

☐ Yes
☒ No




Popout: the breaking away of small portions of a concrete surface due to localized internal pressure which leaves a shallow, typically conical, depression

popouts are not observed

Popout Size

What is the size of the popout?

☐ Small: popouts leaving holes up to 10 mm in diameter, or the equivalent
☐ Large: popouts leaving holes greater than 50 mm in diameter, or the equivalent
☒ None of these apply

none of these apply

Descriptions and photographs courtesy of ACI 201.1 R-92

ENTER DATA

Data Entry

Is the data entered correctly?

☐ Yes - accept data
☐ No - retry and correct

CONTINUE

Spalling

Is concrete spalling evident?


☒ Yes
 ☐ No


Spalling: a fragement, usually in the shape of a flake, detached from a larger mass by a blow, by the action of weather, by pressure, or by expansion within the large

concrete is spalled

Type of Spall

What is the extent of the spalling?

☐ Small: a roughly circular depression not greater than 20 mm in depth or 50 mm in any dimension
 

☒ Large: may be roughly circular or oval or in some cases elongated, more than 20 mm in depth and 150 mm in the greatest dimension
 

☐ None of these apply

a large spalled region is observed

ENTER DATA

Descriptions and photographs courtesy of ACI 201.1 R-92

Data Entry

Is the data entered correctly?

☐ Yes - accept data
 ☐ No - retry and correct

CONTINUE

Spall Distribution

Are the spalled areas uniformly distributed over the entire element under investigation?

☐ Yes, they are spread uniformly

☒ No, they are heavily concentrated in specific zones

☐ These areas are too small to make an assessment on their distribution

the spalled areas are heavily concentrated in specific zones

Spall Size

Approximately what percentage of the element under investigation is spalled?

☐ Less than 2%

☐ Between 2% and 5%

☐ Between 5% and 10%

☒ Between 10% and 15%

☐ Over 15%

☐ Unknown

the spalled areas are between 10% and 15%

ENTER DATA

Data Entry

Is the data entered correctly?

☐ Yes - accept data

☐ No - retry and correct

CONTINUE

ASR

Is ASR (Alkali-Silica Reactivity) occurring?

☐ Yes

☒ No

☐ Not Sure

ASR is not occurring

Three requirements must be met for expansive ASR to occur: (1) reactive forms of silica or silicate in the aggregate; (2) sufficient alkali (sodium and potassium) primarily from the cement; (3) sufficiently available moisture in the concrete. If any one of the three requirements are not met, expansion due to ASR cannot occur.

In its simplest form, ASR can be visualized as a two-step process:

Alkali + Silica --> Gel Reaction Product
Gel Reaction Product + Moisture --> Expansion

Actual expansion occurs in the second step when the ASR gel reaction product swells as it absorbs moisture. Potentially expansive gel reaction product does not form unless the first step

Description courtesy of SHRP-C/FR-91-101

ENTER DATA

Data Entry

Is the data entered correctly?

☐ Yes - accept data

☐ No - retry and correct

CONTINUE

Sulfate Content

What is the sulfate content of the concrete?

☒ Low:
 ☐ Medium:
 ☐ High:
 ☐ Unknown

Source: Concrete Repair and Maintenance Illustrated, Peter Emmons, 1993.

All sulfates are potentially harmful to concrete. They react chemically with cement paste's hydrated lime and hydrated calcium aluminate. As a result of this reaction, solid products with volume greater than the products entering the reaction are formed. The formation of gypsum and ettringite expands, pressurizes and disrupts the paste. As a result, surface scaling and disintegration set in, followed by mass deterioration. Sulphate content samples are gathered and prepared in the same manner as samples from chloride determination. In this case the extraction is carried out using concentrated hydrochloric acid. The solution is neutralized using dilute ammonium hydroxide and then barium chloride is added to produce a precipitate of barium sulphate. The weight of barium sulphate produced permits the sulphate content of the concrete sample to be calculated.

the sulfate content is low

Sulfate Exposure

Is the concrete surface subjected to sulfate-rich soils or liquids?

☐ Yes
 ☐ No
 ☒ Unknown

it is unknown if concrete is exposed to sulfate-rich materials

ENTER DATA

Data Entry

Is the data entered correctly?

☐ Yes - accept data
 ☐ No - retry and correct

CONTINUE

Carbonation Depth

If measured, what is the approximate depth of the carbonation front?

- ☐ Less than 10% of the depth of cover
- ☐ Between 10% and 25% of depth of cover
- ☐ Between 25% and 50% of depth of cover
- ☐ Between 50% and 75% of depth of cover
- ☐ Between 75% and 100% of depth of cover
- ☐ Greater than 100% of depth of cover
- ☒ Unknown

Carbonation of concrete is the reaction among acidic gases from the air, moisture, and the alkaline cement paste. To determine the depth of carbonation, a fresh concrete surface must be exposed. This can be done by core sampling the suspect surface and splitting the core with a hammer and chisel. The position of carbonation front is measured by spraying the concrete surface with an acid-based indicator which changes colors at a pH of about 10, indicating the interface between carbonated and uncarbonated zones. The most commonly used indicator for this purpose is a solution of phenolphthalein, which colors the concrete an intense red (pink) at pH values greater than 10 and is colorless at pH values less than 10. The pH-indicators are not supposed to give the exact pH value of the concrete, but merely to measure the depth of the layer altered by carbonation.

the carbonation depth is unknown

Source: Concrete Repair and Maintenance Illustrated, Peter Emmons, 1993.

ENTER DATA

Data Entry

Is the data entered correctly?

- ☐ Yes - accept data
- ☐ No - retry and correct

CONTINUE

Deicing Salts

Is the bridge element under consideration exposed to deicing salts or subjected to salt spray?

- ☒ Yes
- ☐ No

member is exposed to salt spray

ENTER DATA

Data Entry

Is the data entered correctly?

- ☐ Yes - accept data
- ☐ No - retry and correct

CONTINUE

Chloride Content at Depth of Cover

If measured, is the chloride content of concrete at the depth of cover:

- ☐ Less than 0.05% by weight of cement (acid soluble)
- ☐ Less than 0.05% by weight of cement (water soluble)
- ☐ Between 0.05% and 0.15% by weight of cement (acid soluble)
- ☐ Between 0.05% and 0.15% by weight of cement (water soluble)
- ☐ Between 0.15% and 0.25% by weight of cement (acid soluble)
- ☒ Between 0.15% and 0.25% by weight of cement (water soluble)
- ☐ Between 0.25% and 0.35% by weight of cement (acid soluble)
- ☐ Between 0.25% and 0.35% by weight of cement (water soluble)
- ☐ Greater than 0.35% by weight of cement (acid soluble)
- ☐ Greater than 0.35% by weight of cement (water soluble)
- ☐ Unknown

Chloride testing is done by taking a sample of concrete from the structure, either by drawing pulverized concrete, or by taking cores than pulverizing the concrete in the laboratory. At each level of sampling, the pulverized material is collected and stored in a clean container, the hole is vacuum cleaned, and the next sample is drawn at the next desired depth. Powered samples are analyzed using a wet chemical method.

Source: Concrete Repair and Maintenance Illustrated, Peter Emmons, 1993

the chloride content is between 0.15 and 0.25% (water soluble)

ENTER DATA

Data Entry

Is the data entered correctly?

☐ Yes - accept data

☐ No - retry and correct

CONTINUE

Chloride Content at 50% Cover Depth

If measured, is the chloride content of concrete at 50% of the depth of cover:

- ☐ Less than 0.05% by weight of cement (acid soluble)
- ☐ Less than 0.05% by weight of cement (water soluble)
- ☐ Between 0.05% and 0.15% by weight of cement (acid soluble)
- ☐ Between 0.05% and 0.15% by weight of cement (water soluble)
- ☐ Between 0.15% and 0.25% by weight of cement (acid soluble)
- ☐ Between 0.15% and 0.25% by weight of cement (water soluble)
- ☐ Between 0.25% and 0.35% by weight of cement (acid soluble)
- ☒ Between 0.25% and 0.35% by weight of cement (water soluble)
- ☐ Greater than 0.35% by weight of cement (acid soluble)
- ☐ Greater than 0.35% by weight of cement (water soluble)
- ☐ Unknown

Chloride testing is done by taking a sample of concrete from the structure, either by drawing pulverized concrete, or by taking cores than pulverizing the concrete in the laboratory. At each level of sampling, the pulverized material is collected and stored in a clean container, the hole is vacuum cleaned, and the next sample is drawn at the next desired depth. Powered samples are analyzed using a wet chemical method.

Source: Concrete Repair and Maintenance Illustrated, Peter Emmons, 1993

the chloride content is between 0.25 and 0.35% (water soluble)

ENTER DATA

Data Entry

Is the data entered correctly?

☐ Yes - accept data

☐ No - retry and correct

CONTINUE

Permeability

If measured, is the permeability of concrete:

☐ Low

☐ Medium

☐ High

☒ Unknown

the permeability is unknown

Several types of apparatus have been developed for site use in measuring properties related to permeability. The initial surface absorption test (ISAT) uses a cap sealed to the surface of the concrete under test using modelling clay. When the test is undertaken on a vertical surface, a means of keeping the cap firmly in contact with the surface has to be provided. Water is introduced into the cap to give a pressure head of 200 mm using a filter tunnel. A second port in the cap leads to a horizontal capillary tube. The rate at which water is absorbed into the concrete surface is determined by closing the connection to the reservoir and measuring the movement of water surface in the capillary tube during a fixed time period.

Rapid Chloride Permeability Test:

The chloride content of concrete can be determined determined by analyzing pulverized concrete samples at various depths using Rapid Chloride Test (RCT) 1029. The RCT measures the acid soluble amount of chlorides as a percentage of concrete mass. A specified amount of chloride powder is extracted and mixed with a vial containing 10 mL of extraction liquid. A potential reading is taken with the RCT chloride electrode and then converted to chloride content in percent of concrete weight using the provided calibration chart.

ENTER DATA

Data Entry

Is the data entered correctly?

☐ Yes - accept data

☐ No - retry and correct

CONTINUE

Corrosion Stains

Are corrosion/rust stains evident on the concrete surface?

☒ Yes

☐ No

corrosion stains present

Exposed Steel

Are any reinforcing steel bars/stirrups/tendons exposed?

☒ Yes

☐ No

steel is exposed

Corrosion Products

Are rust stains or corroded rebar evident under spalled areas?

☐ Yes, light rust

☒ Yes, moderate rust

☐ Yes, extensive rust

☐ No

☐ Unknown

moderate corrosion products are observed on steel

ENTER DATA

Data Entry

Is the data entered correctly?

☐ Yes - accept data

☐ No - retry and correct

CONTINUE

Epoxy Coating

Is the steel reinforcement epoxy coated?

☐ Yes

☒ No

☐ Unknown

the steel is not epoxy coated

Application Time

When was the epoxy coated bar used in this member?

☐ Original construction

☐ During reconstruction or repair

☒ Unknown

it is unknown when/if epoxy coated bars were used

ENTER DATA

Data Entry

Is the data entered correctly?

☐ Yes - accept data

☐ No - retry and correct

CONTINUE

Compressive Stength Values

What was the compressive strength of the core or cylinder, if available?

☐ Less than 1500 psi

☐ Between 1500-3500 psi

☐ Between 3500-4000 psi

☐ Between 4000-4500 psi

☐ Between 4500-5000 psi

☐ Between 5000-5500 psi

☐ Between 5500-6000 psi

☐ Between 6000-6500 psi

☒ Unknown

☐ Not applicable (no test data available)

the compressive strength is unknown

ENTER DATA

Data Entry

Is the data entered correctly?

☐ Yes - accept data

☐ No - retry and correct

CONTINUE

Concrete Quality

Based on the information obtained on the member under consideration, please rate the basic quality of the concrete as:

☐ Very good

☐ Good

☒ Average

☐ Marginal

☐ Poor

☐ Unknown

the concrete quality is average

ENTER DATA

Data Entry

Is the data entered correctly?

☐ Yes - accept data

☐ No - retry and correct

CONTINUE

Please click the button to run the analysis:		RUN ANALYSIS	GET RESULT
Diagnosis			
Diagnosis	1	Steel Corrosion - Due to Chlorides from Deicing Salts	
Extent of Damage			
Extent	1	Extensive Damage	
Prescribed Repair			
Possible Corrective Actions 1			
Provide support for the beam as necessary. Remove damaged and spalled concrete and apply a low permeability patch material. Consider the fact that the transfer length of strand has thus been increased by the length of strand exposed during concrete removal.			
To protect the prestressed concrete beam-ends against long-term intrusion of chlorides through leaky expansion joints, apply an appropriate surface treatment to the end regions (approximately 2 ft on each end). Refer to the project report for a summary of the effectiveness of various treatments for this application. This surface treatment should ideally be applied in new construction or as early as possible.			
It is suggested that efforts be made to limit further chloride contamination of concrete.			
It is suggested that the leaky expansion joint be effectively repaired, or consideration be given to retrofitting into a jointless bridge.			
Repair expansion joints or consider retrofitting the bridge into a jointless bridge. A major research report prepared by the Construction Technology Laboratories, Inc. (CTL) for the Federal Highway Administration addresses various issues and procedures for retrofitting jointed into jointless bridges.			

Example 2: Structurally Deficient

Member Observations:

The member under consideration is a pier column located on a 45-year-old two span bridge in Wisconsin. The bridge is located in an urban area with an ADT of over 10,000. The bridge carries a city street and spans over a city street. It is not located near industrial sites. The bridge does carry heavy trucks. There are weight limits posted and the bridge has been classified as structurally deficient, but not functionally obsolete. It has been assigned a rating of three. No vehicle impact damage has occurred. Some previous patch repairs have been performed on the column. More than 8% of the member has been patched with a portland cement patch. The condition of the patch is not very good, but the patches have not spalled yet. It is unknown when the first patch material was placed, but the most recent patch work was completed 8 years ago. The bridge does have leaky expansion joints, but no support settlement is observed.

Extensive cracking and spalling is observed. The cracks are orientated randomly and are less than 12 inches in length. The cracks are not related to extraordinary loading and not related to flexural or shear loading. The cracks have not been observed to move noticeably with temperature changes.

No other surface defects such as honeycombing, blistering, or abrasion is observed. New delamination and spalling areas are noted

No alkali silica reactivity (ASR) is suspected. The sulfate content is low and the member is not subjected to sulfate contaminated soils. The depth of the carbonation front is unknown. The member is exposed to deicing salts. Testing for chloride content has not been performed. The permeability of the concrete is medium. Corrosion stains are observed on the concrete, and no exposed steel is observed. The compressive strength of concrete is unknown. The overall concrete quality can be given a marginal rating.

A printout of the result page of the ConBAR session for the example detailed above is shown below.

Please click the button to run the analysis:		RUN ANALYSIS	GET RESULT
Diagnosis			
Diagnosis	1	Steel Corrosion - Due to Chlorides from Deicing Salts	
Extent of Damage			
Extent	1	Extensive Damage	
Prescribed Repair			
Possible Corrective Actions	1	Consider replacing the member under investigation. Any repair or rehabilitation strategy at this stage of deterioration may be short-lived.	
Structural Deficiency			

Example 3: Light Damage

Member Observations:

The member under consideration is a pier column located on a 14-year-old three span bridge in Wisconsin. The bridge is located in a rural area with an ADT of 2500. The bridge carries a state highway and spans over a county road. It is not located near industrial sites. The bridge does carry heavy trucks. There are no weight limits posted and the bridge has not been classified as structurally deficient or functionally obsolete. It has been assigned a rating of seven. No vehicle impact damage has occurred and no previous repairs have been performed on the beam. The bridge does not have any drainage issues and no support settlement is observed.

Light craze cracking are observed. The cracks are orientated randomly and are less than 12 inches in length. The cracks are not related to extraordinary loading and not related to flexural or shear loading. It is unknown if the crack planes run through the aggregates and no residue is observed around the cracks. The cracks have not been observed to move noticeably with temperature changes.

No other surface defects such as honeycombing, blistering, abrasion, scaling, or popouts are observed. No delaminations are observed. No spalling is observed.

No alkali silica reactivity (ASR) is suspected. The sulfate content is low and the member is not subjected to sulfate contaminated soils. The depth of the carbonation front is unknown. The member is exposed to deicing salts and the acid-soluble chloride content at the depth of cover

is 0.04% by weight of cement. The acid-soluble chloride content at half of the cover depth is 0.01% by weight of cement. The permeability of the concrete is not measured and is unknown. Corrosion stains are not observed on the concrete. No exposed steel or corrosion products are observed. The steel is epoxy coated and the coating was applied prior to the original construction. The compressive strength of concrete is unknown. The overall concrete quality can be rated as good.

A printout of the result page of the ConBAR session for the example detailed above is shown below.

Please click the button to run the analysis:		RUN ANALYSIS	GET RESULT
Diagnosis			
Diagnosis	1	Steel Corrosion - Due to Chlorides from Deicing Salts	
Extent of Damage			
Extent	1	Light Damage	
Prescribed Repair			
Possible Corrective Actions 1 It is suggested that efforts be made to limit further chloride contamination of concrete.			
To protect the concrete against long-term intrusion of chlorides, apply an appropriate surface treatment (penetrating sealers or coatings). Refer to the project report for a summary of previous tests on the effectiveness of various treatments. Such surface treatments are most effective when they are applied early before chloride contamination has progressed.			

APPENDIX B

Specimen Shop Drawings

REPLACEMENT SPANCRETE "I-GIRDER" MATERIAL LIST						SHEET#:	1
PC. #	QTY.	SIZE	LENGTH	WEIGHT	CONCRETE CU./YD.	SPAN#	
1-7607	5	36"	8'-0"	3,280#	0.78 CY	1	

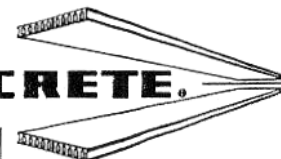
"PRODUCTION NOTES"

1. ALL REBAR DIMENSIONS IN SCHEDULE ARE OUT TO OUT.
2. ALL STRANDS TO BE CUT FLUSH W/ ENDS OF GIRDERS.
3. TOPS OF GIRDERS TO BE ROUGH FLOATED AND BROOMED TRANSVERSLY FOR BONDING-
~ TO THE SLAB, EXCEPT THE OUTSIDE 2" SHALL BE TROWEL FINISHED.
4. ENDS OF STRANDS SHALL BE PAINTED WITH NON-STAINING GRAY NON BITUMINOUS JOINT SEALER.
~ (THIS APPLIES ONLY TO THOSE ENDS OF GIRDER THAT ARE FINALLY EXPOSED).
5. GIRDER DATA: STRANDS:
() 1/2" 270,000 P.S.I LOW-RELAXATION STRANDS (UN-DRAPE) (STRESS TO K)
~ LONG BARS-----FY= 60 KSI ~ CONCRETE STRENGTH @ RELEASE: (4,800) PSI
~ STIRRUPS-----FY= 60 KSI ~ CONCRETE STRENGTH @ 28-DAYS: (6,000) PSI
7. WELDED WIRE FABRIC SHALL CONFORM TO ASTM A497.

SPANCRETE INC.

2448 CENTURY ROAD
GREEN BAY, WI.54307-0608

SPANCRETE.



TEST BEAMS
UNIVERSITY OF WISCONSIN-
MILWAUKEE

STA.# :

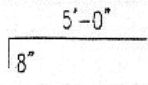


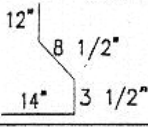
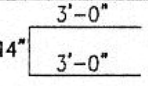

STRUCTURE # :



SECTION # :

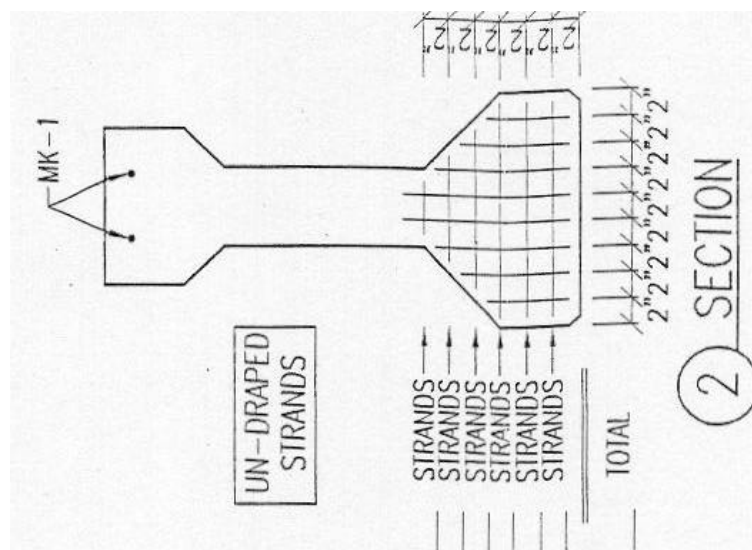
PROJECT # :

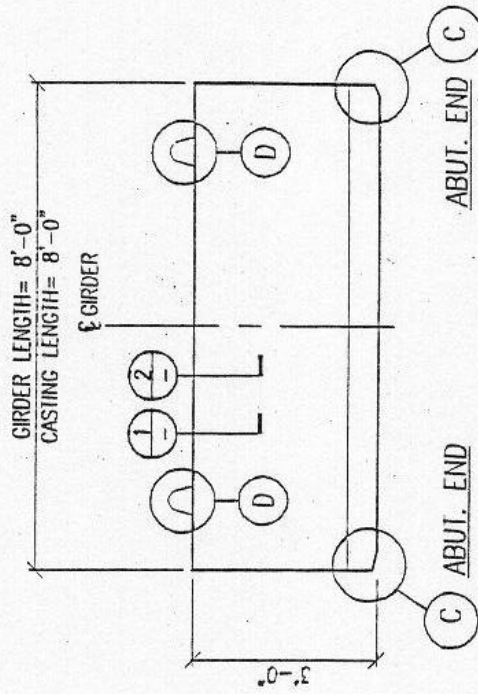
COUNTY :

ENGINEER:	DRAWN BY: KRISTI	DATE: 12-12-01
CONTRACTOR:	JOB CAPTAIN: TP-9211	CHECKED: <i>URC</i> 12/14/01
APPROVED:	REVISED: _____	JOB NO.: 1-7607
DRAWING FILE NO.: H:\DWG1\1-7607\SH01.DWG	TASK# 2402	SHEET: 1 OF 1

CAST IN HARDWARE					
MK.#	QTY.	LENGTH	SIZE	SHAPE	REMARKS
1 4-0508	20	5'-8"	#4		(2) REQ'D @ TOP EACH END
3 5-0306	20	3'-6"	#5		(2) REQ'D @ EA. END
4 5-0308	40	3'-8"	#5		HALF RIGHT HALF LEFT
6 3-0302	140	3'-2"	#3		(7) LOC. EA. END. (EPOXY COATED) (USED IN PAIRS)
7 4-0702	10	7'-2"	#4		(1) REQ'D @ EA. END @ BTM.
9 4-0700	10	7'-0"	#4		(1) REQ'D @ EA. END @ TOP

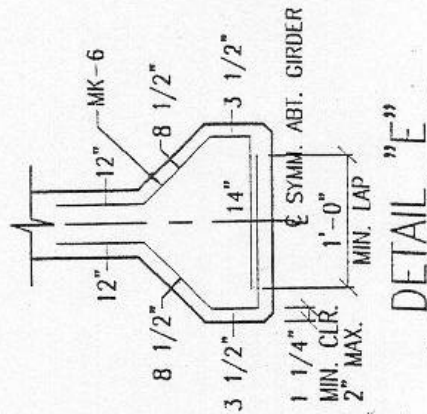
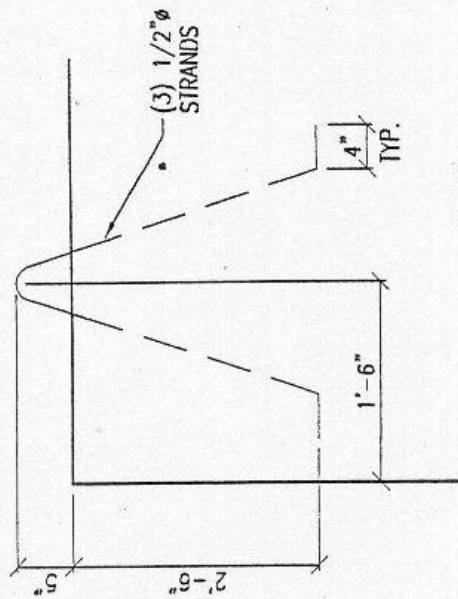
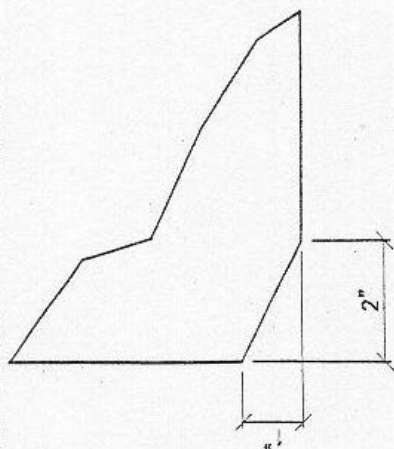
20 4-0404	152	4'-3 1/2"	#4		U-SHAPED EPOXY COATED
36-4	154 SF	SEE DESIGN	W7.5xD18		U-SHAPED BENT (SEE DESIGN DETAIL)
0481603	LF 1/2"Ø 270 KSI LOW-RELAXATION (7) WIRE STRAND				





STRAND PROFILE-- STRAIGHT

STIRRUP & MESH DESIGN



APPENDIX C

Spancrete Test Reports



SALES OFFICE: P.O. BOX 10508 • 2448 CENTURY ROAD • GREEN BAY, WI 54307-0508 • 920 / 494-0274 • FAX 920 / 494-7901

February 15, 2002

DR. HABIB TABATABAI, PH.D., P.E., S.E.
UNIVERSITY OF WISCONSIN-MILWAUKEE
DEPARTMENT OF CIVIL ENGINEERING & MECHANICS
3200 N. CRAMER ST.
MILWAUKEE WI 53211

Enclosed are the 28-day test reports for the (5) 36" I beams we made for you.

If you have any questions or would like additional information, please contact me at (920) 965.9451.

Sincerely,

Michael Grochowski
Quality Control

MG:sd
enc

please reply to ADMINISTRATIVE OFFICE unless indicated below

SPANCRETE, INC.

☐ PLANT

P.O. Box 188
Valders, Wisconsin 54245
920 / 775-4121
Fax 920 / 775-4511

☐ SALES OFFICE

P.O. Box 828
Waukesha, WI 53187-0828
414 / 290-9000
Fax 414 / 290-9125

SPANCRETE INDUSTRIES, INC.

☐ PLANT

1600 East Main Street
Waukesha, Wisconsin 53186
414 / 290-9000 • Fax 414 / 290-9170
Engineering
414 / 290-9100 • Fax 414 / 290-9120

SPANCRETE OF ILLINOIS, INC.

☐ PLANT & SALES OFFICE

4012 Route 14
Crystal Lake, Illinois 60014
815 / 459-5580
773 / 775-6402
Fax 815 / 459-0510

SPECIMEN TYPES: *Concrete*

- 0 - Cylinder
- 1 - Cube
- 2 - Beam, Center Point Loading
- 3 - Beam, Third Point Loading, Fracture in Middle Third of Span
- 4 - Beam, Third Point Loading, Fracture Outside of Middle Third
- 5 - Cross Sectional Area

Cylinder Break Type per ASTM C39

- 1 - Cone
- 2 - Cone and Split
- 3 - Cone and Shear
- 4 - Shear
- 5 - Columnar

DATE	TIME	OP ID#	SP TYP	SP ID#	AREA (sq in)	LENGTH (in)	PK LOAD (Lb)	PK STRESS (Lb/sq in)	C.F.	BRK TYPE
1	01-08-02	07:06	11	074700211	12.566	8.000	59120.	4704.	1.000	4
2	01-08-02	07:09	11	074700221	12.566	8.000	56740.	4515.	1.000	4
3	01-08-02	07:13	11	073370811	12.566	8.000	51590.	4105.	1.000	4
4	01-08-02	12:26	11	074145581	12.566	8.000	89780.	7144.	1.000	4
5	01-08-02	12:29	11	074142683	12.566	8.000	94800.	7543.	1.000	4
6	01-08-02	12:32	11	074790581	12.566	8.000	101470.	8074.	1.000	4
7	01-09-02	07:07	11	074700311	12.566	8.000	56880.	4526.	1.000	4
8	01-09-02	07:09	11	074700321	12.566	8.000	53890.	4288.	1.000	4
9	01-09-02	07:14	11	073370911	12.566	8.000	43710.	3478.	1.000	4
10	01-09-02	07:31	11	074170111	12.566	8.000	65450.	5208.	1.000	4
11	01-09-02	08:07	11	072531311	12.566	8.000	56560.	4500.	1.000	4
12	01-09-02	08:35	11	073370912	12.566	8.000	51020.	4060.	1.000	4
13	01-09-02	13:03	11	074145681	12.566	8.000	96330.	7665.	1.000	4
14	01-09-02	13:07	11	074142783	12.566	8.000	101690.	8092.	1.000	4
15	01-09-02	13:10	11	075430181	12.566	8.000	87640.	6974.	1.000	4
16	01-09-02	13:14	11	072530581	12.566	8.000	95530.	7602.	1.000	4
17	01-10-02	07:09	11	074700411	12.566	8.000	65140.	5183.	1.000	4
18	01-10-02	07:12	11	074700421	12.566	8.000	60860.	4843.	1.000	4
19	01-10-02	07:16	11	073371011	12.566	8.000	67060.	5336.	1.000	4
20	01-10-02	07:19	11	074140211	12.566	8.000	45400.	3612.	1.000	4
21	01-10-02	08:21	11	076070111	28.274	12.000	188730.	6674.	1.000	4
22	01-10-02	08:24	11	076070112	28.274	12.000	184430.	6522.	1.000	4
23	01-10-02	12:30	11	074145781	12.566	8.000	100540.	8000.	1.000	4
24	01-10-02	12:33	11	074142883	12.566	8.000	95790.	7622.	1.000	4
25	01-10-02	12:36	11	075430281	12.566	8.000	96180.	7653.	1.000	4
26	01-10-02	15:13	11	072530681	12.566	8.000	104180.	8290.	1.000	4

RELEASE

STATISTICS: 26 Samples

Low:	43710.	3478.
Mean:	83481.	6008.
High:	188730.	8290.
+3 Sigma:	193484.	10946.
-3 Sigma:	-26521.	1071.

SPECIMEN TYPES:

0 - Cylinder

1 - Cube

Beam, Center Point Loading

3 - Beam, Third Point Loading, Fracture in Middle Third of Span

4 - Beam, Third Point Loading, Fracture Outside of Middle Third

5 - Cross Sectional Area

Cylinder Break Type per ASTM C39

1 - Cone

2 - Cone and Split

3 - Cone and Shear

4 - Shear

5 - Columnar

DATE	TIME	OP ID#	SP TYP	SP ID#	AREA (sq in)	LENGTH (in)	PK LOAD (Lb)	PK STRESS (Lb/sq in)	C.F.	BRK TYPE
1	02-05-02	07:06	11	074140411	12.566	8.000	39400.	3135.	1.000	4
2	02-05-02	07:10	11	074144931	12.566	8.000	57380.	4566.	1.000	4
3	02-05-02	08:12	11	074140211	12.566	8.000	58990.	4694.	1.000	4
4	02-05-02	08:16	11	076560411	28.274	12.000	145300.	5138.	1.000	4
5	02-05-02	08:20	11	076560421	28.274	12.000	139810.	4944.	1.000	4
6	02-05-02	08:33	11	074140412	12.566	8.000	45090.	3588.	1.000	4
7	02-05-02	12:00	11	074700381	12.566	8.000	110280.	8775.	1.000	4
8	02-05-02	12:21	11	074700382	12.566	8.000	105100.	8363.	1.000	4
9	02-05-02	12:24	11	072531381	12.566	8.000	111970.	8910.	1.000	4
10	02-05-02	12:27	11	073370981	12.566	8.000	90450.	7197.	1.000	4
11	02-05-02	16:52	11	074170181	12.566	8.000	98610.	7847.	1.000	4
12	02-06-02	07:08	11	074140511	12.566	8.000	59880.	4765.	1.000	4
13	02-06-02	07:11	11	074144931	12.566	8.000	57040.	4539.	1.000	4
14	02-06-02	08:19	11	076560511	28.274	12.000	151760.	5367.	1.000	4
15	02-06-02	08:22	11	076560521	28.274	12.000	136740.	4836.	1.000	4
16	02-06-02	15:18	11	076070181	12.566	8.000	111300.	8856.	1.000	4
17	02-06-02	15:21	11	076070182	12.566	8.000	110330.	8779.	1.000	4
18	02-06-02	15:28	11	076070183	12.566	8.000	111550.	8876.	1.000	4
19	02-06-02	15:50	11	074700481	12.566	8.000	107930.	8588.	1.000	4
20	02-06-02	17:03	11	074700482	12.566	8.000	97610.	7767.	1.000	4
21	02-06-02	17:04	11	073371081	12.566	8.000	107070.	8520.	1.000	4
22	02-06-02	17:06	11	074170281	12.566	8.000	88940.	7077.	1.000	4
23	02-07-02	07:14	11	074145111	12.566	8.000	60190.	4789.	1.000	4
24	02-07-02	07:18	11	0 2060211	12.566	8.000	60230.	4792.	1.000	4
25	02-07-02	08:16	11	076560611	28.274	12.000	165300.	5846.	1.000	4
26	02-07-02	08:20	11	076560621	28.274	12.000	174680.	6178.	1.000	4
27	02-07-02	08:44	11	0 0	12.566	8.000	66470.	5289.	1.000	4
28	02-07-02	14:55	11	074143683	12.566	8.000	100200.	7973.	1.000	4
29	02-07-02	14:58	11	074141982	12.566	8.000	92430.	7355.	1.000	4
30	02-07-02	15:07	11	074170381	12.566	8.000	91260.	7262.	1.000	4
31	02-07-02	15:12	11	073371181	12.566	8.000	94480.	7518.	1.000	4
32	02-07-02	15:14	11	072531481	12.566	8.000	90430.	7196.	1.000	4

STATISTICS: 32 Samples

Low:	39400.	3135.
Mean:	98068.	6541.
High:	174680.	8910.
+3 Sigma:	201082.	11884.
-3 Sigma:	-4945.	1199.

THIS MEMORANDUM

is an acknowledgment that a bill of lading has been issued and is not the Original Bill of Lading, nor a copy of duplicate, covering the property named herein, and is rendered solely for filing or record.

RECEIVED CONTRACTOR CLASSIFICATIONS AND TARIFFS IN EFFECT ON THE DATE OF THE RECEIPT BY THE CARRIER OF THE PROPERTY DESCRIBED IN THE ORIGINAL BILL OF LADING.

DATE: 9/10/2001

16226

SHIPPER NO.

P.O. NO. 2176713

BOL #: 22690-1

FROM: SHIPPER

TO: CONSIGNEE

SPARKRETE INDUSTRIES
2440 CENTURY RD.
GREEN BAY WI 54303
(414) 790-8000

STRUCTURAL REINFORCEMENT PROD.
309-A FOREST ROAD
HIMMELST INDUSTRIAL PARK
HAZLETON, PA 18201

No. Packages	Description Of Articles, Special Marks And Exceptions	Weight Subject To Correction
100	95-SPA-011239-36-4 VK4-D7.5/D18 (GRADE 70) 66'' (+0.5, +0.5)X12'' (1.1) LS (0.5'' ON, 102, 1024, 102, 1010, 102, 0.5'' ON)	24599 10,192
100	95-SPA-011239-36-8 VK8-D7.5/D18 (GRADE 70) 66'' (+0.5, +0.5)X12'' (1.1) LS (0.5'' ON, 102, 1024, 102, 1010, 102, 0.5'' ON)	24591 8,961
150	95-SPA-011239-36-17 VK17-D7.5/D18 (GRADE 70) 66'' (+0.5, +0.5)X14'' (1.1) LS (0.5'' ON, 102, 1024, 102, 1010, 102, 0.5'' ON)	24592 10,008
150	95-SPA-011239-45-8 VK8-D7.5/D18 (GRADE 70) 84'' (+0.5, +0.5)X12'' (1.1) LS (0.5'' ON, 102, 1032, 102, 1010, 102, 0.5'' ON)	24594 16,955
TOTAL WEIGHT		45,216

SPARKRETE INC.
GREEN BAY
RECEIVED

DATE:

SIGN:

RECEIVED
GREEN BAY
SPARKRETE INC.

I Subject to Section 7, Conditions of applicable bill lading, if this Shipment is to be delivered to the consignee, I, Consignee, shall sign the below statement.

The carrier shall not be delivery of the shipment with payment of freight and other bills charged.

SIGNATURE OF CONSIGNEE

If charges are in prepaid, or on stamp form, "to be prepaid"

Received \$ _____
I hereby acknowledge the charges, and the property described herein.

Agent or Carrier

Per _____
(The signature here acknowledges only the amount prepaid.)

Charges Advanced

\$ _____

I Shipper's receipt is this stamp and a part of bill of lading approved by the Interstate Commerce Commission.

I The first lines used for it shipper's receipt so it specifications and both in the to make a certificate thereof, and other requirements of Consular Freight Classification.

If the shipment moves between ports by a carrier by water, the carrier requires that the bill of lading of state whether it is carrier's shipper's weight.

NOTE - Where the rate depends on value, shipper is required to state specifically making the agreed or declared value of the property.

The agreed or declared value of the property is hereby specified as stated by the shipper to be:

According _____

per _____

If the shipment moves between two ports by a carrier by water, the law requires that the bill of lading shall state whether it is "carrier's or shipper's weight". NOTE - Where the rate is dependent on value, shippers are required to state specifically in writing the agreed or declared value of the property. The agreed or declared value of the property is hereby specifically stated by the shipper to be not exceeding _____

STRUCTURAL REINFORCEMENT PRODUCTS

REFS RUSCO

Agent.

Per TA Permanent post-office address of shipper, HAZLETON, PA 18201

Per TA

RENDER FREIGHT BILLS IN DUPLICATE

1 of 1

SEP 26 2001

3

REBAR AND MESH REPORT

Job Number: 1-7607

Job Name: 36" IB

[illegible]

STRESSING DATA - SPANCRETE GREEN BAY

Inspector: M. GRUCHOWSKI

Job No.: 1-7607-2402 Structure No. UW MILWAUKEE Project No.: _____
 Bed: 36" I-BEAM Total Stress fsi: _____ No. Strand: _____
 Strand Manufacturer: SWPC Grade: 270K Diameter: 1/2
 Reel No.: DM 12746 DM 17751 Heat No.: D242715
 Area (A): .1528 Mod Elasticity (E): 28400 000
 Load Per Strand (Pi): 31000 Lbs Bed Len Inches (L): 2625
 Initial Preload (Pp): 3000 Lbs. Pi + Group I lbs (Pt): 31000 + 826 = 31826

Group I Losses (Added Force)

Chuck Slippage: 1/2 f = 826
 Thermal: _____
 Anchor Movement: _____
 Other: _____

Group II (Added Elongation)

Strand Splices: See Pour Sheet
 Dead End Seating: _____
 Other: _____

$$\frac{.5 \times .1528 \times 28400 000}{2625} = 826$$

Concrete Temp: \pm (stressing temp greater than 25° difference to be corrected 1% per 10°)

Thermal = (Degree difference greater than 25°) + 10° = % correction factor.

(Product len. inches + bed len.) x Pi x correction factor = (Thermal load pounds).

$$\frac{(Pt - Pp) \times L}{A \times E} = \text{Elongation}$$

$$\text{Group I Losses} = \frac{(\text{Total Inches}) \times A \times E}{L}$$

A x E

L

$$\frac{40 \times 12}{2625} = 18\% \times 31000 \times .025 = 140 + 31826 = 31966$$

$$\frac{(31966 - 3000) \times 2625}{.1528 \times 28400 000} = 17.52 + 1/8 = 17 \frac{5}{8} \text{ met } \boxed{17 \frac{1}{8} \text{ met}} \begin{matrix} \text{min } 16 \frac{1}{4} \\ \text{max } 18 \end{matrix}$$

$$3000 + 826 + 14 = 3840$$

STRUCTURAL REINFORCEMENT PRODUCTS **QUALITY CONTROL DEPARTMENT** **MILL CERTIFICATION REPORT**

CUSTOMER NAME:	SPANCRETE IND.	ORDER #:	22680	CURRENT DATE:	10/23/01
PROJECT NUMBER:	01-1239	LOT#:	4590	DATE TESTED:	09/20/01
PRODUCT CODE:	95-SPA-011239-36-4			SAMPLES REQUIRED:	1
SHEET DESIGNATION:	V x 4 D 7.5 D 18.0	60 " x 8.17 "	GRADE: 70	SHEET QUANTITY:	100

THE PROPERTIES OF THE MATERIAL ORDERED WERE TESTED FOR COMPLIANCE WITH ASTM SPECIFICATIONS A496 AND A497 WITH THE FOLLOWING RECORDED RESULTS:

LONGITUDINAL SAMPLE	(7.5)	1	2	3	4	AVG	NOMINAL	RESULT
HEAT NO:	125885							
WEIGHT PER FOOT(LBS):		0.269	0.000	0.000	0.000	0.269	0.255	PASS
YIELD LOAD(LBS):		0	0	0	0	0	5250	N/A
YIELD STRENGTH(KSI):		0.0	0.0	0.0	0.0	0.0	70.0	N/A
TENSILE LOAD(LBS):		7246	0	0	0	7246	6000	PASS
TENSILE STRENGTH(KSI):		96.6	0.0	0.0	0.0	96.6	80.0	PASS
DEFORMATION HEIGHT(IN):		0.0260	0.0000	0.0000	0.0000	0.0260	0.0139	PASS
BEND TEST:								PASS

TRANSVERSE SAMPLE	(18.0)	1	2	3	4	AVG	NOMINAL	RESULT
HEAT NO:	906349							
WEIGHT PER FOOT(LBS):		0.578	0.000	0.000	0.000	0.578	0.611	PASS
YIELD LOAD(LBS):		0	0	0	0	0	12600	N/A
YIELD STRENGTH(KSI):		0.0	0.0	0.0	0.0	0.0	70.0	N/A
TENSILE LOAD(LBS):		15231	0	0	0	15231	14400	PASS
TENSILE STRENGTH(KSI):		84.6	0.0	0.0	0.0	84.6	80.0	PASS
DEFORMATION HEIGHT(IN):		0.028	0.000	0.000	0.000	0.0280	0.024	PASS
BEND TEST:								PASS
WELD SHEAR(PSI)		6492	7398	6316	6738	6736	6300	PASS

STATEMENT

THE UNDERSIGNED CERTIFIES THAT THE ABOVE IS A TRUE AND ACCURATE REPRESENTATION OF THE TEST RESULTS OBTAINED ON THE DESCRIBED MATERIAL AS APPEARING ON COMPANY RECORDS. THIS DOCUMENT CERTIFIES THAT THE MATERIAL MENTIONED ABOVE MEETS THE REQUIRED ASTM SPECIFICATIONS AND THAT ALL MATERIAL WAS PROCESSED WITHIN THE UNITED STATES OF AMERICA.

THE MATERIAL TESTED DOES ☒ DOES NOT ☐ CONFORM WITH THE ABOVE STATEMENT

REGARDS,


 DAVID MUNDIE
 QUALITY CONTROL

Denote Plant of Origin:

DATE: 11310140 1
09/25/01

B.L. NO.: 55-I57848
 LOAD NO.: 5520023957
 972 EAST 4500 N ROAD
 BOUBONNAIS, IL 60914-4127
 (815) 937-3131

• OLD
• TO
• AMBASSADOR STEEL CORP
• PO BOX 2340
• KOKOMO, IN 46904-2340
•

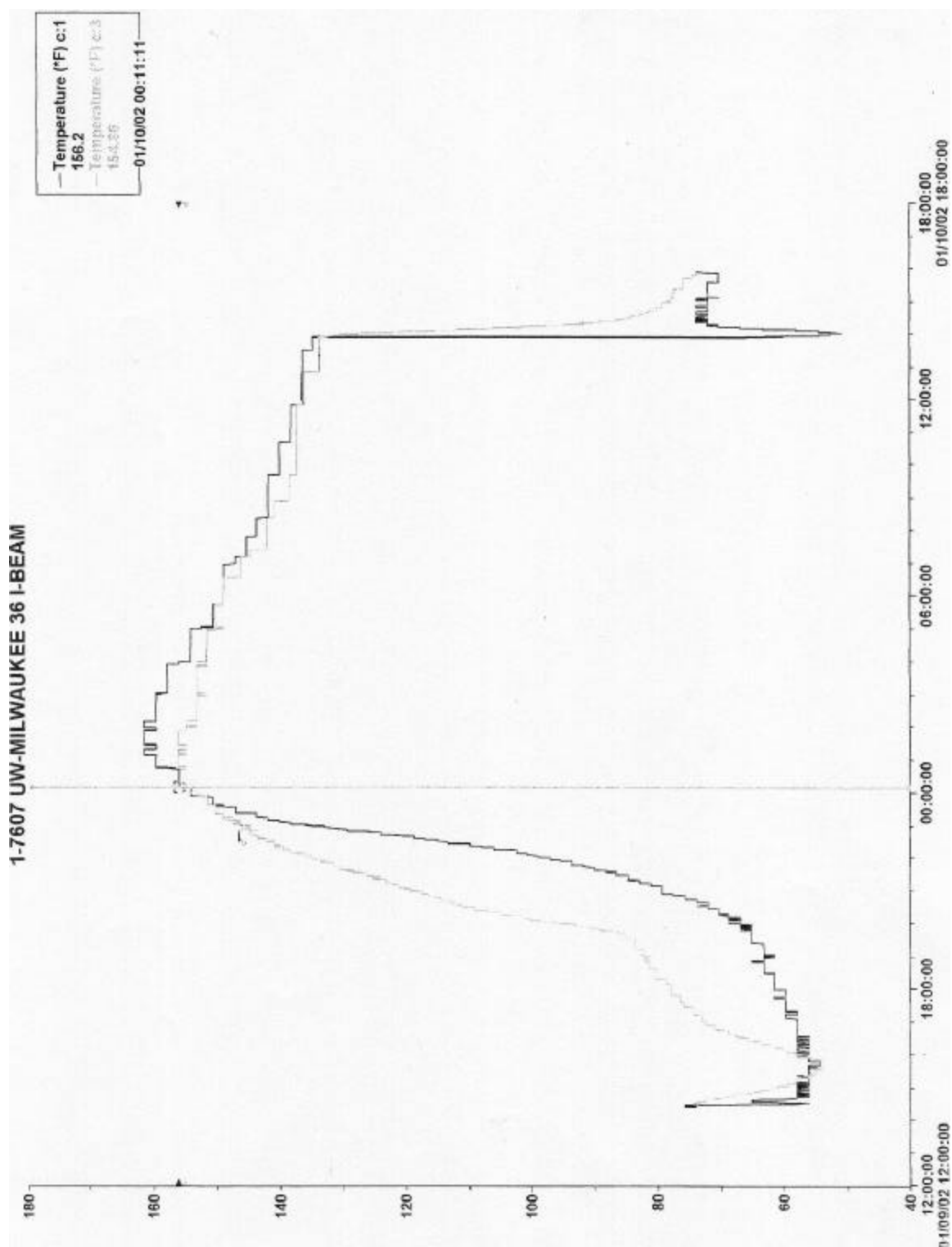
SHIP • AMBASSADOR STEEL CORP
TO • CUST TRUCK
• KOKOMO, IN

HEAT NO.*	DESCRIPTION	PHYSICAL TESTS				CHEMICAL TESTS									
		YIELD P.S.I.	TENSILE P.S.I.	ELONG % IN 8"	Bend	WT% DEF	C	Mn		P	S	V	Si	Cu	C.E.
								Ni	Cr						
CO#: BB23957	KANKAKEE, IL STEEL DIVISION	71,500	111,700	12.0%	OK	-3.7%		.09	1.00	.017	.037	.19	.36		
5502I1617	13 Rebar 60' (Gr420/#4Gr60)	493MPa	770MPa			.038		.15	.20	.07	.001	.001			
	ASTM A615/A615M-96a, GR 420														
5502I1618	KANKAKEE, IL STEEL DIVISION	70,000	109,200	10.0%	OK	-3.1%									
10-4-01	13 Rebar 60' (Gr420/#4Gr60)	483MPa	753MPa			.038									
	ASTM A615/A615M-96a, GR 420														
5502I1080	KANKAKEE, IL STEEL DIVISION	69,300	105,100	13.0%	OK	-4.4%		.39	1.02	.009	.053	.22	.42		
	16 Rebar 60' (Gr420/#5Gr60)	478MPa	725MPa			.038		.14	.12	.07	.013	.001			
	ASTM A615/A615M-96a, GR 420														
5502I1081	KANKAKEE, IL STEEL DIVISION	68,500	103,500	14.0%	OK	-3.9%		.39	.99	.009	.053	.26	.35		
10-4-01	16 Rebar 60' (Gr420/#5Gr60)	472MPa	714MPa			.039		.13	.12	.07	.013	.001			
	ASTM A615/A615M-96a, GR 420														
CE 1080 =	$.39 + \frac{1.92}{6} = .56$	100% per heat required													
CE 1081 =	$.39 + \frac{1.92}{6} = .555$	100% per heat required													
		GREEN BAY RECEIVED													
		DATE: 9-26-01													
		SIGN: TJB													

WE HEREBY CERTIFY THAT THE ABOVE FIGURES ARE CORRECT AS CONTAINED IN THE RECORDS OF THE CORPORATION. MANUFACTURING PROCESSES OF THE STEEL MATERIALS IN THIS PRODUCT, INCLUDING MELTING, HAVE OCCURRED WITHIN THE UNITED STATES, IN COMPLIANCE WITH THE "BUY AMERICAN" PROVISIONS OF THE SURFACE TRANSPORTATION ASSISTANCE ACT OF 1982.

GLEN MCADAM

QUALITY ASSURANCE



APPENDIX D

Chloride Test Data

Measurement of the acid soluble chloride content of hardened concrete by the RCT method

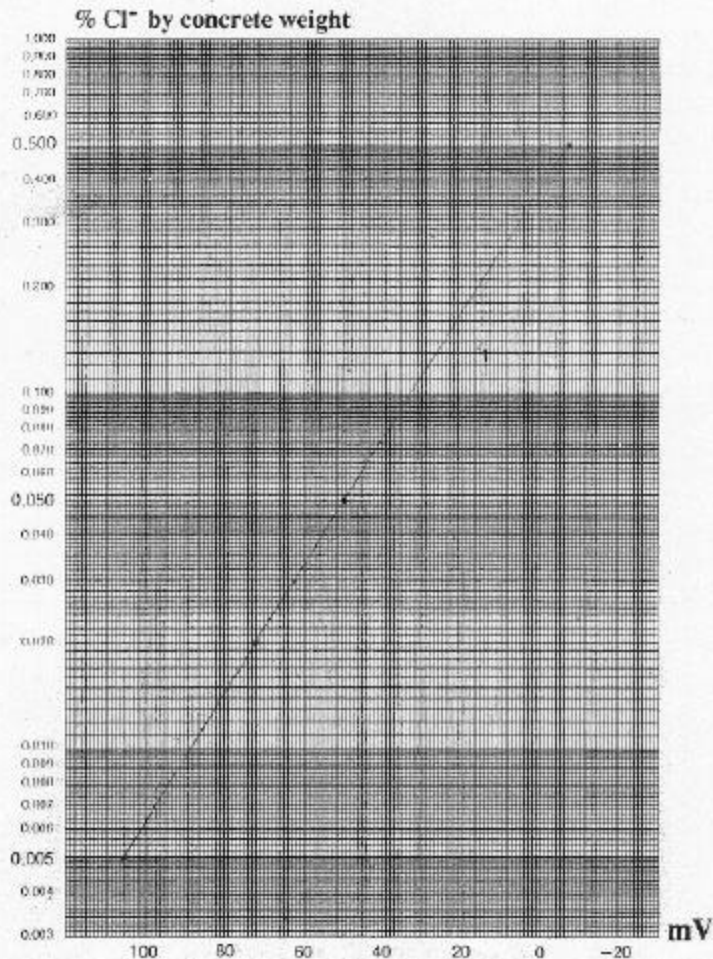
2/19/2002

Report #: _____ Structure: _____ Project: _____
 Date of testing: _____ Electrode #: _____ Person: _____
 Testing Lab: 185 Address: _____ Phone: _____

RCT

**HARDENED
CONCRETE**

1.5 gram of concrete
dust dissolved in a
RCT-1023 vial with
10 milliliter of ex-
traction liquid



CALIBRATION:

Liquid	Clear	Purple	Green	Pink
% Cl	0.005	0.020	0.050	0.500
mV before	105	72.2	50.1	-7.2
mV after				

SAMPLE #	1		2		Remarks
	mV	% Cl	mV	% Cl	
1/4"	62.5	0.030			
1/2"	54.8	0.041			
3/4"	40.5	0.072			
7/8"	55.3	0.040			

Measurement of the acid soluble chloride content of hardened concrete by the RCT method

Report #: 2119/2002 Structure: _____ Project: _____

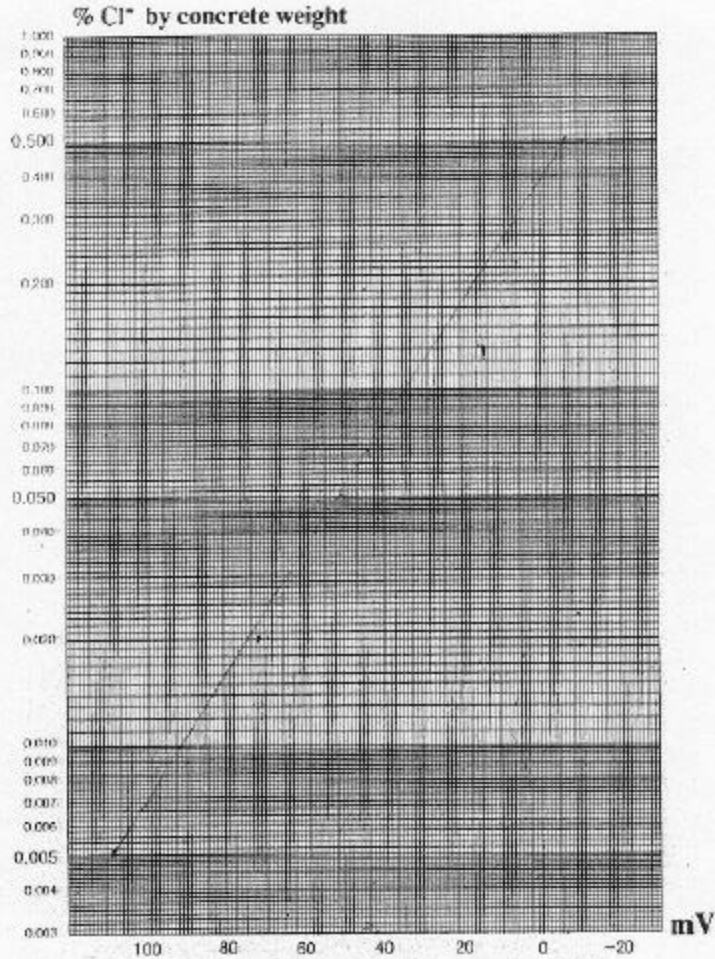
Date of testing: _____ Electrode #: _____ Person: _____

Testing Lab: WZL 9 Address: _____ Phone: _____

RCT

**HARDENED
CONCRETE**

1.5 gram of concrete
dust dissolved in a
RCT-1023 vial with
10 milliliter of ex-
traction liquid



CALIBRATION:

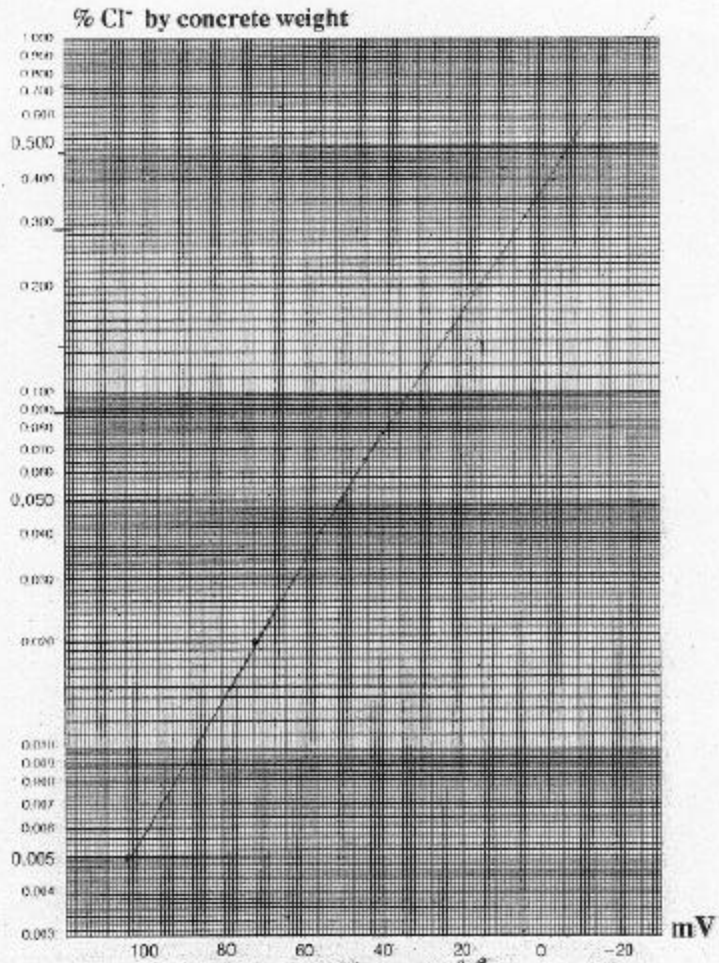
Liquid	Clear	Purple	Green	Pink
% Cl	0.005	0.020	0.050	0.500
mV before	103.9	71.7	50.5	-6.4
mV after				

SAMPLE #	1		2		Remarks
	mV	% Cl	mV	% Cl	
1/4"	57.2	0.040			
1/2"	46.5	0.061			
3/4"	44.1	0.069			
7/8"	43.3	0.070			

Measurement of the acid soluble chloride content of hardened concrete by the RCT method

Report #: _____ Structure: Beam 2-Path Project: _____
 Date of testing: 11/18 Electrode #: _____ Person: _____
 Testing Lab: _____ Address: _____ Phone: _____

RCT
HARDENED CONCRETE
 1.5 gram of concrete dust dissolved in a RCT-1023 vial with 10 milliliter of extraction liquid



CALIBRATION:

Liquid	Clear	Purple	Green	Pink
% Cl	0.005	0.020	0.050	0.500
mV before	104.6	72.5	50.5	-6.8
mV after				

SAMPLE #	1		2		Remarks
	mV	% Cl	mV	% Cl	
2" → 1/4	-24.3	0.96			
1/2	-18.1	0.74			
3/4	-6.4	0.47			
1"	6.5	0.29			
6" → 1/4	13.5	0.21			
1/2	6.5	0.29			
3/4	36.1	0.136			
1"	25.6	0.135			

from face of beam

Measurement of the acid soluble chloride content of fresh concrete by the RCT method

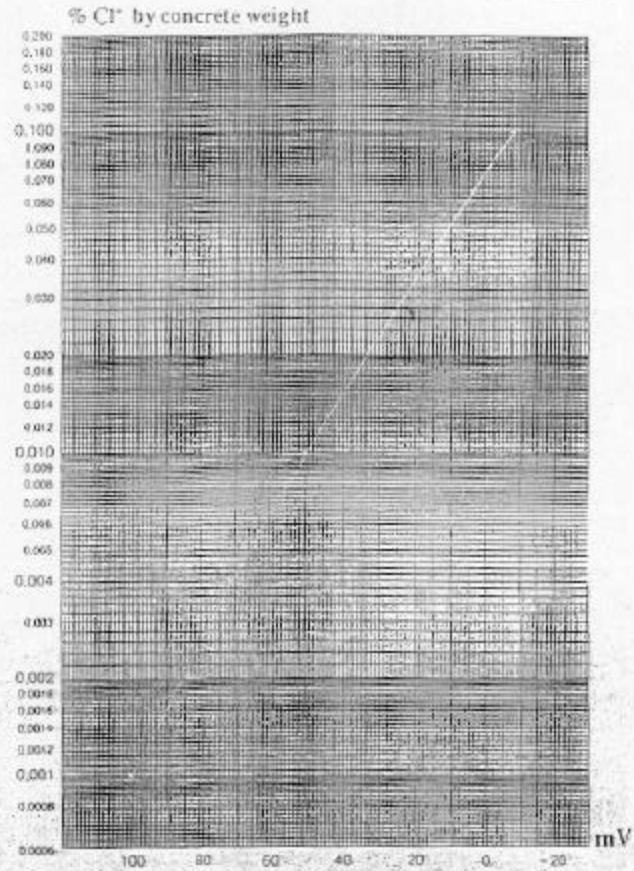
Report #: _____ Mix Identification: _____ Project: Water Test
 Date of testing: 7/17 Electrode #: _____ Person: _____
 Testing Lab: _____ Address: _____ Phone: _____

tested with
 300 mL H₂O

RCT

**FRESH
 CONCRETE**

0.1 liter of fresh concrete mixed into 290 milliliter of RCT extraction liquid



CALIBRATION:

Liquid	Clear	Purple	Green	Pink
% Cl	0.001	0.004	0.010	0.100
mV before	100.7	71.5	50.1	-7.4
mV after				

SAMPLE #	1		2		Remarks
	mV	% Cl	mV	% Cl	
Green Bay	90.2	0.00165	90.3	0.00165	
Milwaukee	93.7	0.0014	93.5	0.0014	

Measurement of the acid soluble chloride content of hardened concrete by the RCT method

Report #: _____ Structure: Beam 1 SW Project: _____

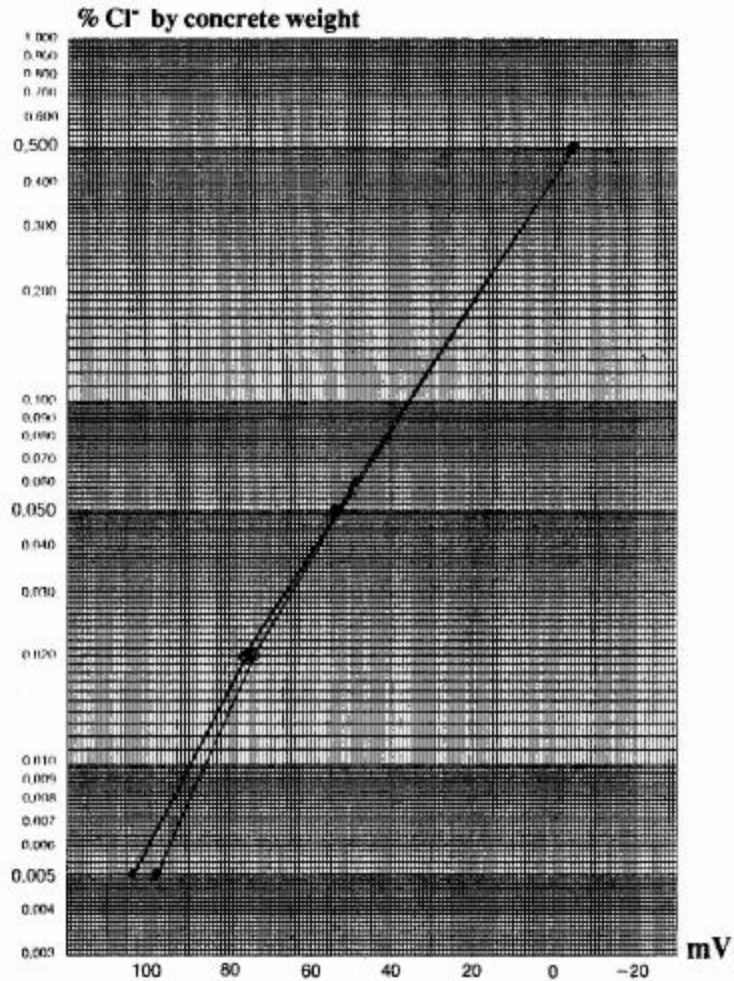
Date of testing: 12-30-2003 Electrode #: _____ Person: AB

Testing Lab: _____ Address: _____ Phone: _____

RCT

**HARDENED
CONCRETE**

1.5 gram of concrete
dust dissolved in a
RCT-1023 vial with
10 milliliter of ex-
traction liquid



CALIBRATION:

Liquid	Clear	Purple	Green	Pink
% Cl	0.005	0.020	0.050	0.500
mV before	98.1	74.3	53.1	-4.8
mV after	103.6	76.1	54.2	-4.1

SAMPLE #	1		2		Remarks
	mV	% Cl	mV	% Cl	
1	43.5	0.071	49.1	0.059	1/4 - 3/4
2	49.0	0.058	49.3	0.058	3/4 - 5/4
3	49.5	0.057	49.2	0.058	5/4 - 7/4
4	41.2	0.080	41.1	0.081	7/4 - 1 1/4
5	48.8	0.070	43.9	0.072	1 1/4 - 1 3/4
6	42.5	0.075	42.3	0.076	1 3/4 - 1 1/2

Measurement of the acid soluble chloride content of hardened concrete by the RCT method

Report #: _____ Structure: Beam 1 SE Project: _____

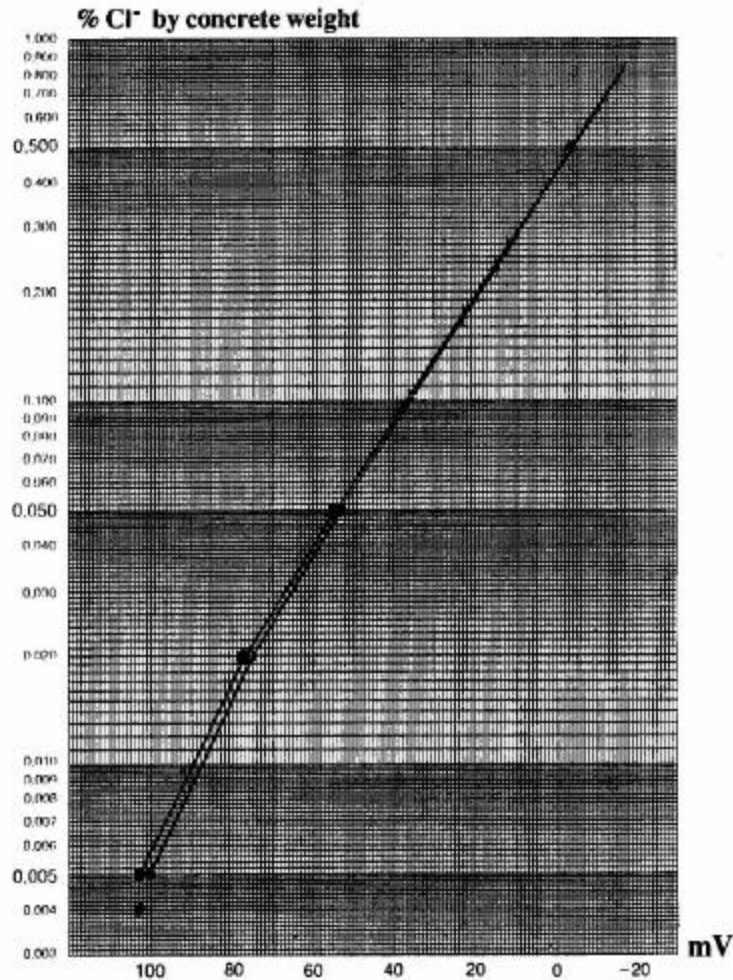
Date of testing: 12-30-2003 Electrode #: _____ Person: CAB

Testing Lab: _____ Address: _____ Phone: _____

RCT

**HARDENED
CONCRETE**

1.5 gram of concrete
dust dissolved in a
RCT-1023 vial with
10 milliliter of ex-
traction liquid



CALIBRATION:

Liquid	Clear	Purple	Green	Pink
% Cl	0.005	0.020	0.050	0.500
mV before	102.9	77.4	54.5	-3.4
mV after	100.4	74.9	53.0	-3.5

SAMPLE #	1		2		Remarks
	mV	% Cl	mV	% Cl	
1	-15.3	0.78	-11.9	0.630	1/8 - 3/8
2	-8.8	0.62	-9.9	0.640	3/8 - 5/8
3	15.1	0.24	14.5	0.240	5/8 - 7/8
4	11.4	0.26	10.1	0.285	7/8 - 1 1/8
5	23.2	0.17	20.2	0.190	1 1/8 - 1 3/8
6	35.8	0.105	35.0	0.103	1 3/8 - 1 5/8

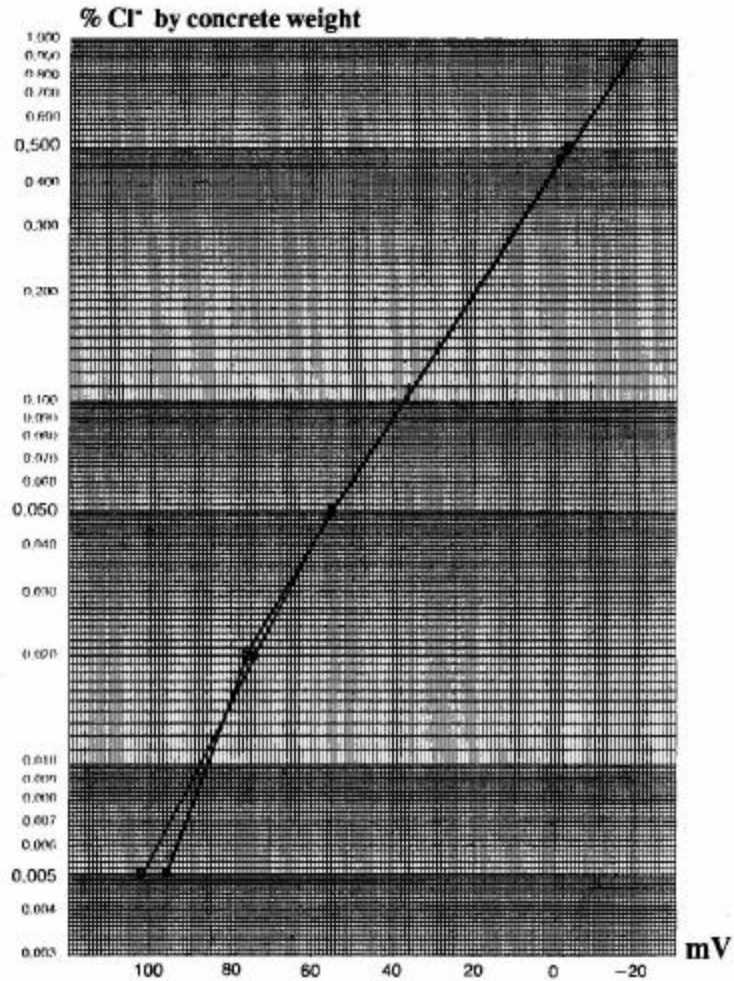
Measurement of the acid soluble chloride content of hardened concrete by the RCT method

Report #: _____ Structure: Beam 2 SW Project: _____
 Date of testing: 12-30-2003 Electrode #: _____ Person: CAB
 Testing Lab: _____ Address: _____ Phone: _____

RCT

**HARDENED
CONCRETE**

1.5 gram of concrete
dust dissolved in a
RCT-1023 vial with
10 milliliter of ex-
traction liquid



CALIBRATION:

Liquid	Clear	Purple	Green	Pink
% Cl	0.005	0.020	0.050	0.500
mV before	101.7	73.9	54.4	-2.7
mV after	96.1	76.0	54.2	-2.8

SAMPLE #	1		2		Remarks
	mV	% Cl	mV	% Cl	
1	16.8	0.83	15.2	0.79	1/8 - 3/8
2	0.9	0.46	1.1	0.42	3/8 - 5/8
3	19.5	0.205	18.0	0.215	5/8 - 1 1/8
4	36.0	0.105	36.6	0.102	7/8 - 1 1/8
5	28.6	0.140	28.0	0.145	1 1/8 - 1 3/8
6	37.1	0.100	37.1	0.100	1 3/8 - 1 5/8

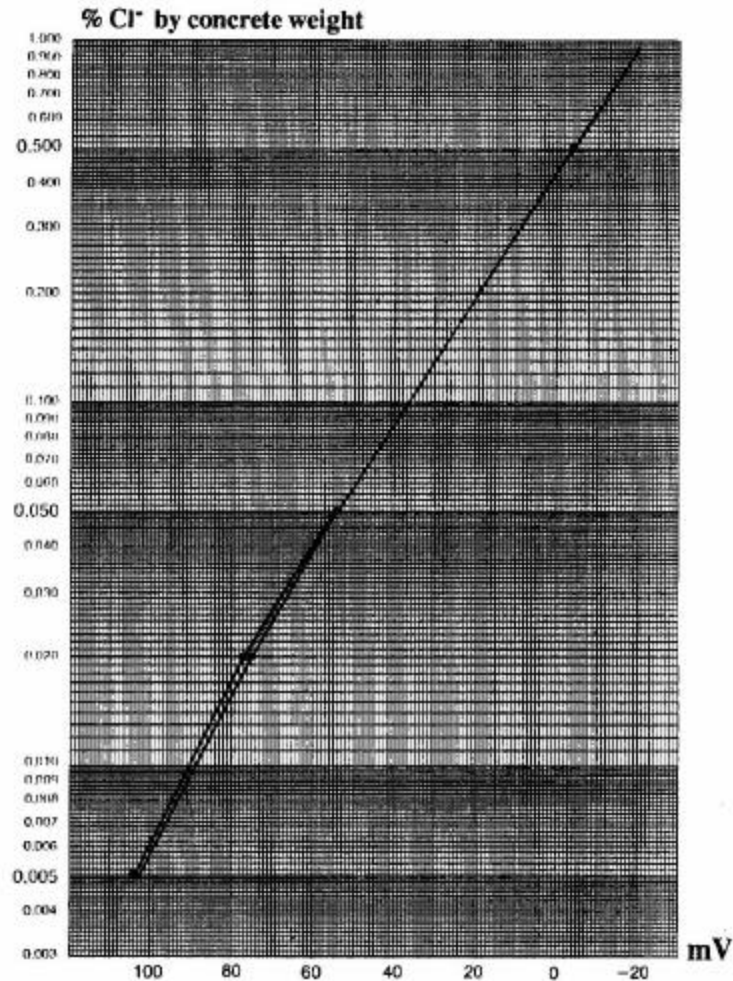
Measurement of the acid soluble chloride content of hardened concrete by the RCT method

Report #: _____ Structure: Beam 2 SE Project: _____
 Date of testing: 1-7-2003 Electrode #: _____ Person: CAB
 Testing Lab: _____ Address: _____ Phone: _____

RCT

**HARDENED
CONCRETE**

1.5 gram of concrete
dust dissolved in a
RCT-1023 vial with
10 milliliter of ex-
traction liquid



CALIBRATION:

Liquid	Clear	Purple	Green	Pink
% Cl	0.005	0.020	0.050	0.500
mV before	103.7	76.8	54.7	-3.4
mV after	102.9	75.2	54.0	-3.0

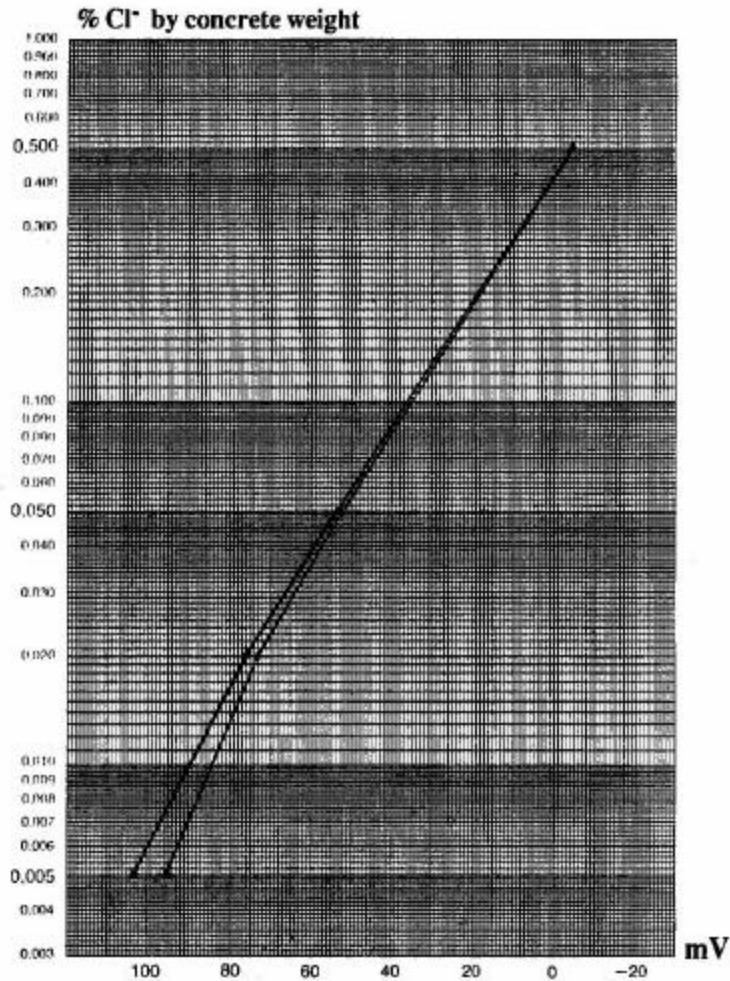
SAMPLE #	1		2		Remarks
	mV	% Cl	mV	% Cl	
1	-2.5	0.475	-2.3	0.475	1/8 - 3/8
2	3.4	0.375	4.4	0.360	3/8 - 5/8
3	-5.2	0.520	-5.5	0.530	5/8 - 7/8
4	-9.5	0.620	-10.4	0.650	7/8 - 1 1/8
5	-10.5	0.620	-8.8	0.600	1 1/8 - 1 3/8
6	-4.3	0.510	-5.1	0.520	1 3/8 - 1 5/8

0.70
0.75
0.80
1.01
1.31
1.10

Measurement of the acid soluble chloride content of hardened concrete by the RCT method

Report #: _____ Structure: Beam 3 SW Project: _____
 Date of testing: 1-7-2003 Electrode #: _____ Person: CAB
 Testing Lab: _____ Address: _____ Phone: _____

RCT
HARDENED CONCRETE
 1.5 gram of concrete dust dissolved in a RCT-1023 vial with 10 milliliter of extraction liquid



CALIBRATION:

Liquid	Clear	Purple	Green	Pink
% Cl	0.005	0.020	0.050	0.500
mV before	103.2	76.3	53.4	-3.9
mV after	95.4	73.3	52.4	-3.7

SAMPLE #	1		2		Remarks
	mV	% Cl	mV	% Cl	
1	29.1	0.132	26.6	0.143	1/8 - 5/8
2	48.7	0.061	42.1	0.074	3/8 - 5/8
3	42.4	0.077	39.0	0.084	5/8 - 7/8
4	45.7	0.068	38.9	0.083	7/8 - 1 1/8
5	55.6	0.046	48.8	0.057	1 1/8 - 1 5/8
6	39.2	0.088	41.7	0.076	1 3/8 - 1 5/8

1.310

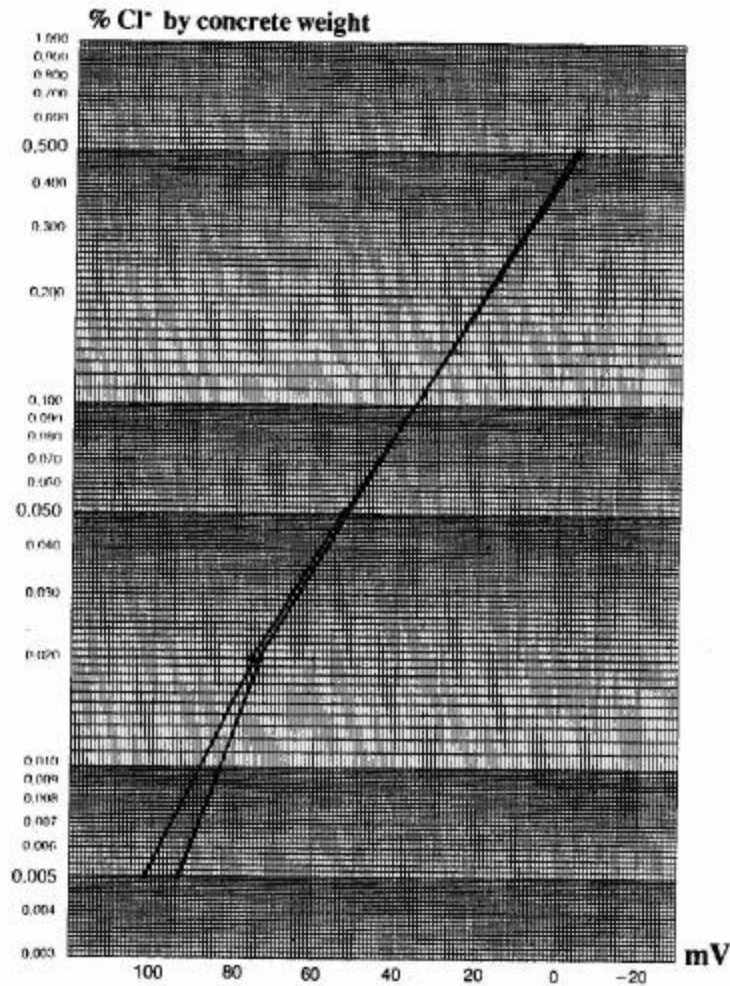
Measurement of the acid soluble chloride content of hardened concrete by the RCT method

Report #: _____ Structure: Beam 3 SE Project: _____
 Date of testing: 1-7-2003 Electrode #: _____ Person: CAB
 Testing Lab: _____ Address: _____ Phone: _____

RCT

**HARDENED
CONCRETE**

1.5 gram of concrete
dust dissolved in a
RCT-1023 vial with
10 milliliter of ex-
traction liquid



CALIBRATION:

Liquid	Clear	Purple	Green	Pink
% Cl	0.005	0.020	0.050	0.500
mV before	101.6	75.8	53.2	-3.7
mV after	93.6	73.9	52.7	-2.6

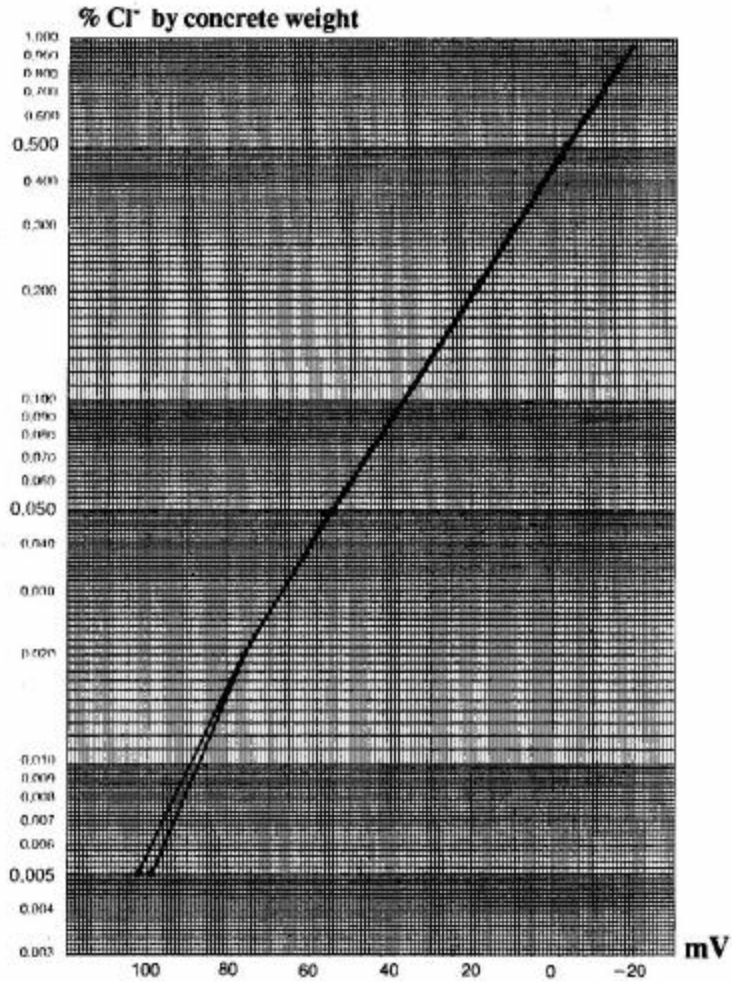
SAMPLE #	1		2		Remarks
	mV	% Cl	mV	% Cl	
1	0.5	0.430	1.3	0.430	1/8 - 3/8
2	18.4	0.225	18.9	0.205	3/8 - 5/8
3	34.2	0.105	34.5	0.105	5/8 - 7/8
4	26.9	0.142	26.8	0.145	7/8 - 1 1/8
5	33.5	0.108	33.2	0.110	1 1/8 - 1 3/8
6	29.3	0.127	34.5	0.105	1 3/8 - 1 5/8

1.18
1.21

Measurement of the acid soluble chloride content of hardened concrete by the RCT method

Report #: _____ Structure: Beam 4 SW Project: _____
 Date of testing: 1-7-2003 Electrode #: _____ Person: CAB
 Testing Lab: _____ Address: _____ Phone: _____

RCT
HARDENED CONCRETE
 1.5 gram of concrete dust dissolved in a RCT-1023 vial with 10 milliliter of extraction liquid



CALIBRATION:

Liquid	Clear	Purple	Green	Pink
% Cl	0.005	0.020	0.050	0.500
mV before	102.1	76.4	53.8	-3.1
mV after	79.0	76.3	54.7	-2.6

SAMPLE #	1		2		Remarks
	mV	% Cl	mV	% Cl	
1	-16.1	0.840	-18.6	0.870	1/8 - 3/8
2	-3.1	0.500	-3.4	0.510	3/8 - 5/8
3	5.7	0.350	5.6	0.360	5/8 - 7/8
4	11.3	0.275	10.5	0.295	7/8 - 1 1/8
5	-1.6	0.470	-1.6	0.480	1 1/8 - 1 3/8
6	6.6	0.335	6.5	0.340	1 3/8 - 1 5/8

1.39

1.07

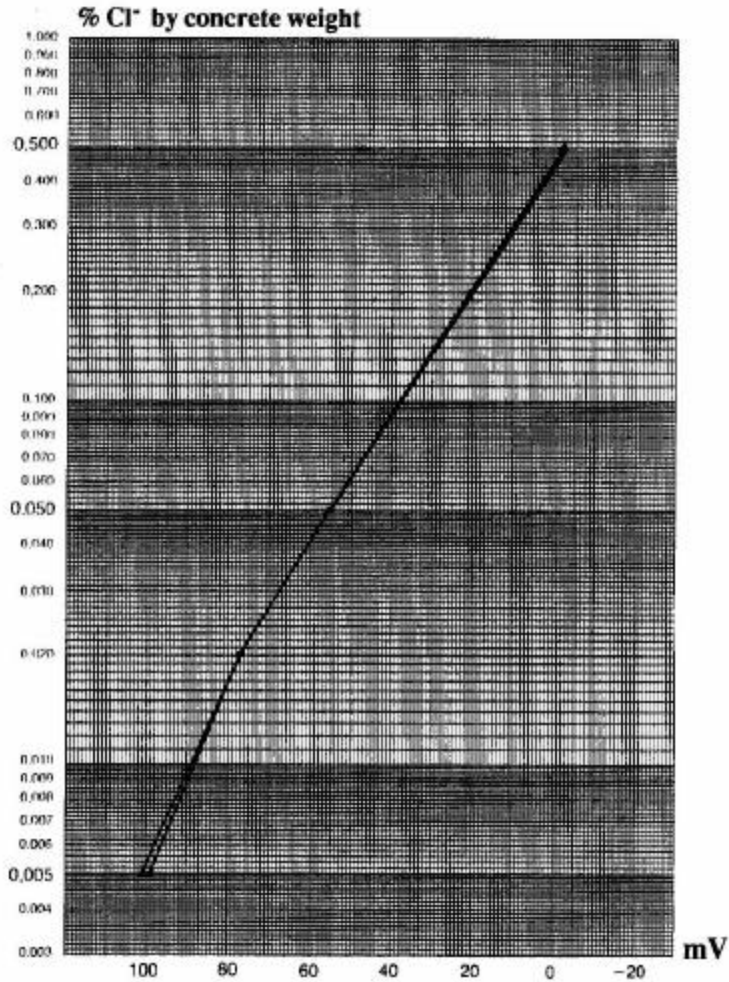
Measurement of the acid soluble chloride content of hardened concrete by the RCT method

Report #: _____ Structure: Beam 4 SE Project: _____
 Date of testing: 1-7-2003 Electrode #: _____ Person: CAB
 Testing Lab: _____ Address: _____ Phone: _____

RCT

**HARDENED
CONCRETE**

1.5 gram of concrete
dust dissolved in a
RCT-1023 vial with
10 milliliter of ex-
traction liquid



CALIBRATION:

Liquid	Clear	Purple	Green	Pink
% Cl	0.005	0.020	0.050	0.500
mV before	99.0	77.4	55.6	-1.9
mV after	100.7	77.7	55.3	-2.9

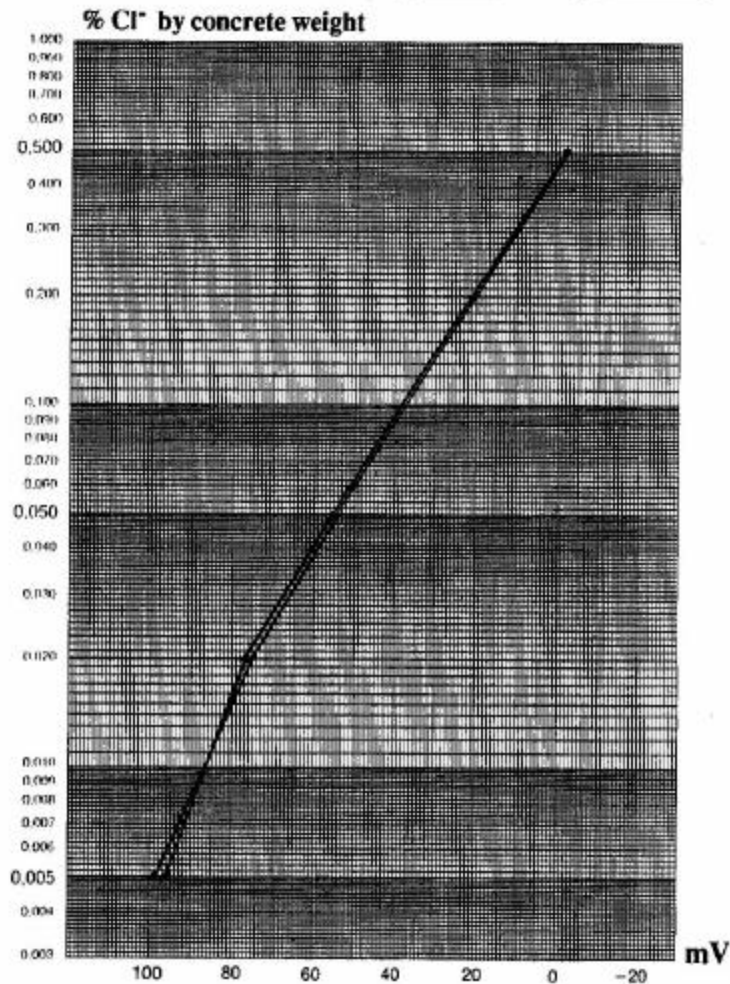
SAMPLE #	1		2		Remarks
	mV	% Cl	mV	% Cl	
1	12.7	0.275	14.5	0.250	1/8 - 3/8
2	8.8	0.320	8.6	0.520	3/8 - 5/8
3	11.8	0.295	11.0	0.290	5/8 - 7/8
4	35.3	0.118	32.3	0.123	7/8 - 1 1/8
5	35.8	0.115	32.9	0.120	1 1/8 - 1 3/8
6	36.2	0.105	35.1	0.100	1 3/8 - 1 5/8

1.25
1.52

Measurement of the acid soluble chloride content of hardened concrete by the RCT method

Report #: _____ Structure: Beam 5 SW Project: _____
 Date of testing: 1-7-2003 Electrode #: _____ Person: CAB
 Testing Lab: _____ Address: _____ Phone: _____

RCT
HARDENED CONCRETE
 1.5 gram of concrete dust dissolved in a RCT-1023 vial with 10 milliliter of extraction liquid



CALIBRATION:

Liquid	Clear	Purple	Green	Pink
% Cl	0.005	0.020	0.050	0.500
mV before	95.9	77.0	55.6	-1.9
mV after	98.7	75.0	53.7	-2.7

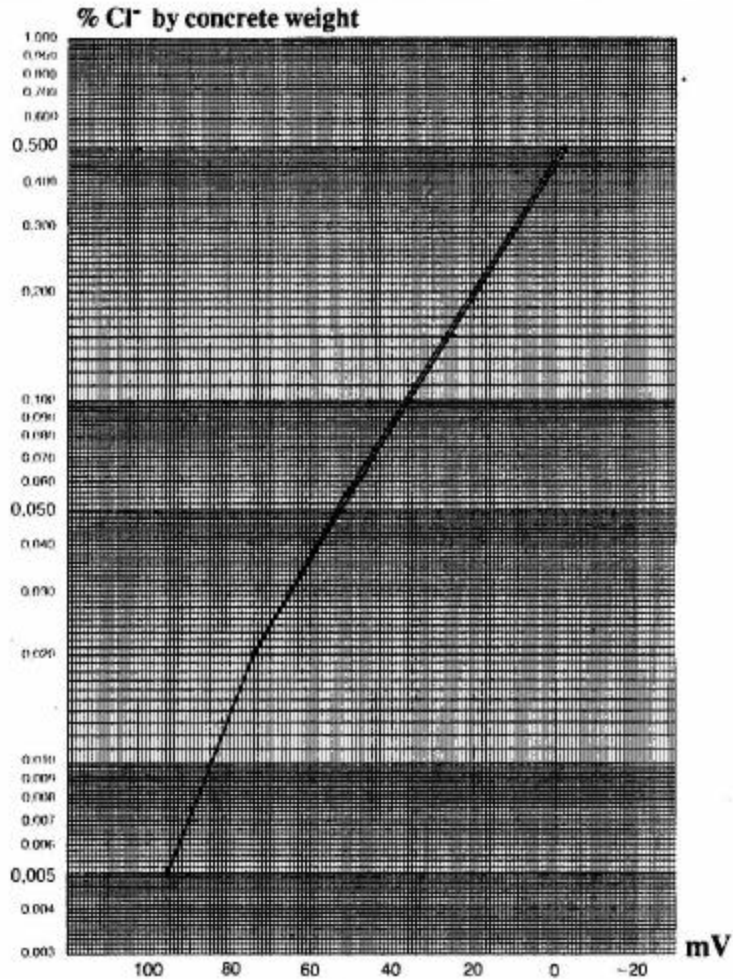
SAMPLE #	1		2		Remarks
	mV	% Cl	mV	% Cl	
1	53.9	0.053	49.9	0.060	1/8 - 3/8
2	45.0	0.076	44.0	0.076	3/8 - 5/8
3	50.8	0.060	50.3	0.060	5/8 - 7/8
4	53.9	0.053	49.4	0.061	7/8 - 1 1/8
5	46.1	0.072	45.1	0.073	1 1/8 - 1 3/8
6	45.0	0.076	44.4	0.076	1 3/8 - 1 5/8

1.46

Measurement of the acid soluble chloride content of hardened concrete by the RCT method

Report #: _____ Structure: Beam 5 SE Project: _____
 Date of testing: 1-7-2003 Electrode #: _____ Person: AB
 Testing Lab: _____ Address: _____ Phone: _____

RCT
HARDENED CONCRETE
 1.5 gram of concrete dust dissolved in a RCT-1023 vial with 10 milliliter of extraction liquid



CALIBRATION:

Liquid	Clear	Purple	Green	Pink
% Cl	0.005	0.020	0.050	0.500
mV before	95.6	74.7	52.6	-2.6
mV after	95.4	74.4	53.5	-1.7

SAMPLE #	1		2		Remarks
	mV	% Cl	mV	% Cl	
1	46.6	0.064	46.7	0.068	1/8 - 7/8
2	50.6	0.054	50.0	0.058	7/8 - 5/8
3	49.3	0.056	48.5	0.062	5/8 - 7/8
4	46.7	0.062	45.6	0.070	7/8 - 1 1/8
5	31.5	0.052	51.2	0.055	1 1/8 - 1 3/8
6	35.3	0.100	36.2	0.100	1 3/8 - 1 5/8

1.22
1.27

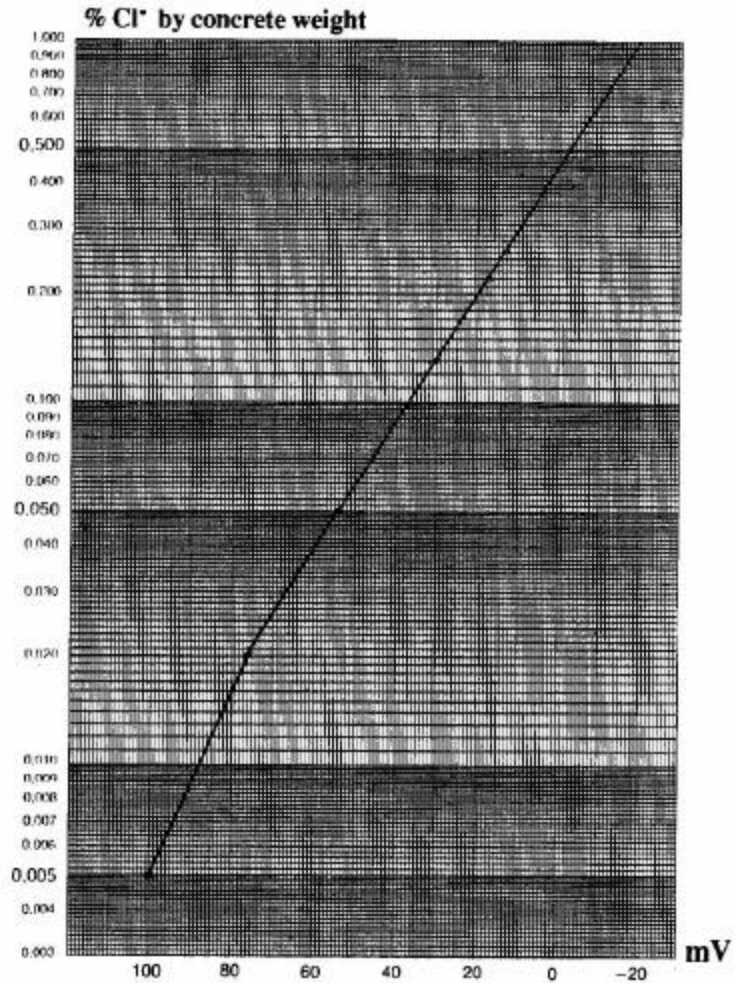
Measurement of the acid soluble chloride content of hardened concrete by the RCT method

Report #: _____ Structure: Beam checks Project: _____
 Date of testing: 1-9-2003 Electrode #: _____ Person: CA B
 Testing Lab: _____ Address: _____ Phone: _____

RCT

**HARDENED
CONCRETE**

1.5 gram of concrete
dust dissolved in a
RCT-1023 vial with
10 milliliter of ex-
traction liquid



CALIBRATION:

Liquid	Clear	Purple	Green	Pink
% Cl	0.005	0.020	0.050	0.500
mV before	100.1	76.3	54.3	-2.4
mV after				

SAMPLE #	1		2		Remarks Sample #
	mV	% Cl	mV	% Cl	
B1A-1	45.2	0.072	40.0	0.090	B3A-6
B1B-1	-12.2	0.740	17.6	0.225	B3B-2
B1B-6	34.9	0.110	30.2	0.133	B3B-6
B2A-1	-15.7	0.840	13.1	0.265	B4B-1
B2A-2	-0.6	0.465			
B2A-4	34.5	0.110			
B2B-2	4.5	0.375			
B2B-5	-8.0	0.620			

Measurement of the acid soluble chloride content of hardened concrete by the RCT method

Report #: Waterford Structure: 3/4" (resin) kg. Project: _____

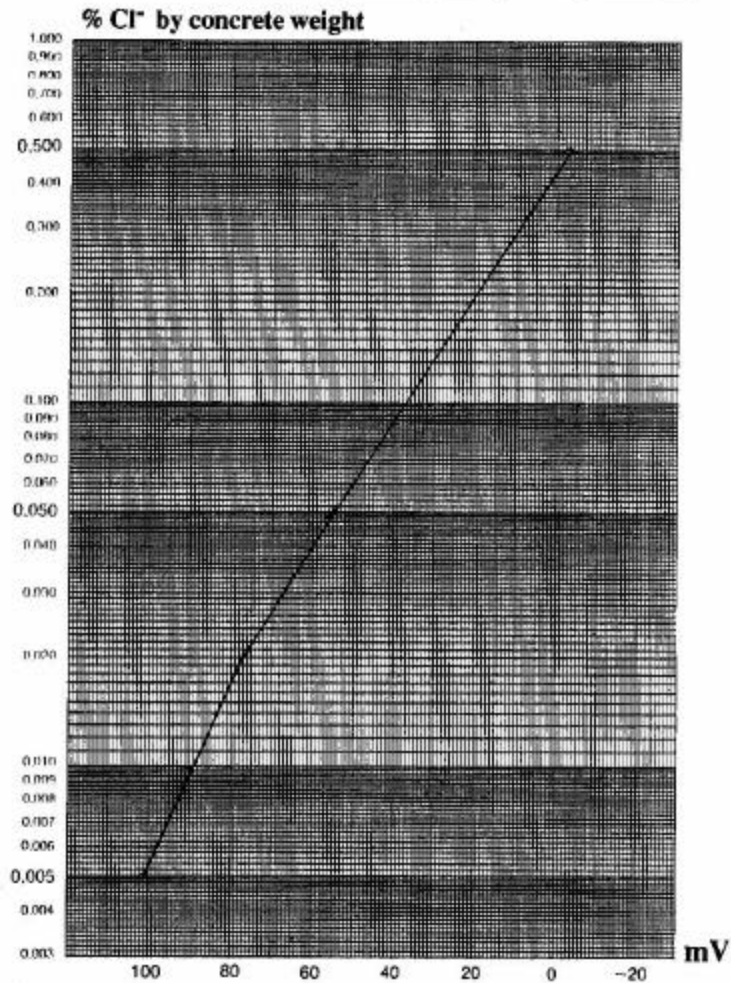
Date of testing: 1-16-04 Electrode #: _____ Person: CAB

Testing Lab: _____ Address: _____ Phone: _____

RCT

**HARDENED
CONCRETE**

1.5 gram of concrete
dust dissolved in a
RCT-1023 vial with
10 milliliter of ex-
traction liquid



CALIBRATION:

Liquid	Clear	Purple	Green	Pink
% Cl	0.005	0.020	0.050	0.500
mV before	101.1	77.3	54.8	-2.8
mV after				

SAMPLE #	1		2		Remarks
	mV	% Cl	mV	% Cl	
1	62.9	0.035			3/4" (resin) kg. Green

5-min test

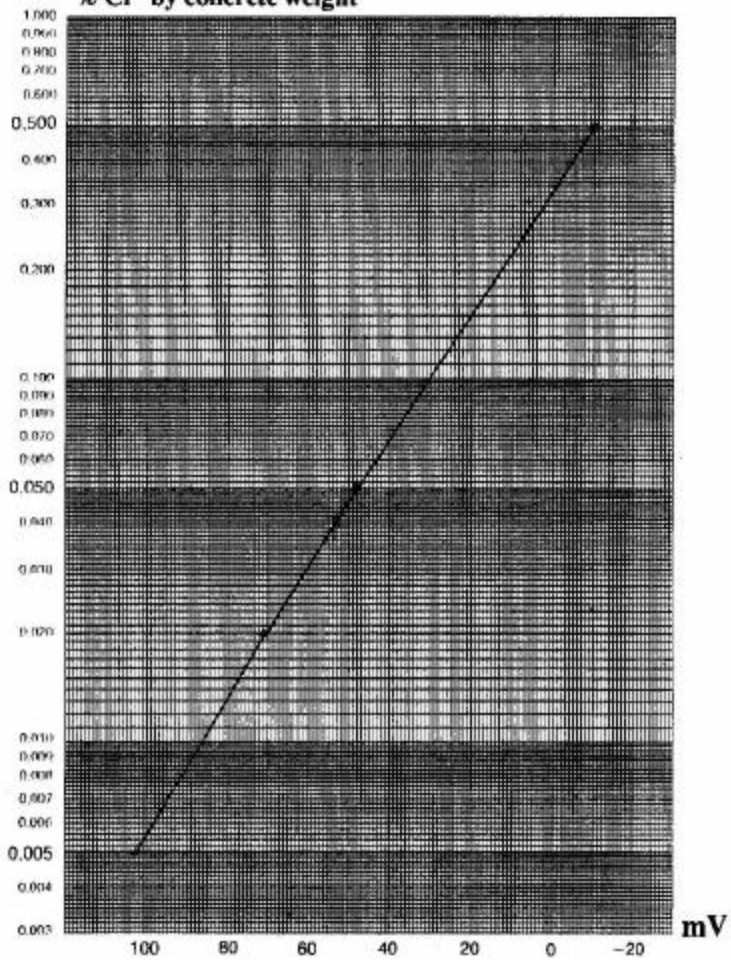
Report #: _____ Structure: _____ Project: For Prof. Takahashi

Date of testing: 07/14/03 Electrode #: _____ Person: KL

Testing Lab: _____ **Address:** _____ **Phone:** _____

HARDENED CONCRETE

% Cl⁻ by concrete weight



Liquid	Clear	Purple	Green	Pink
% Cl	0.005	0.020	0.050	0.500
mV before				
mV after	102.4	71.0	47.9	-9.7

[illegible]

APPENDIX E

Steel Loss Calculations

Constant (g/C)

$$\text{Con} := 3.6 \left[\frac{(55.847)}{(2.96487)} \right] \left(\frac{1}{10^3} \right) \cdot 3600 = 3.6$$

BEAM 1 $\text{Con} = 1.042 \times 10^{-3}$

$$\text{B1E1} := \int_0^{5138.4208} 100.2646 \exp(-0.0002738t) \, dt$$

$$\text{B1E1} = 2.765 \times 10^5 \text{ C}$$

$$\text{Con} \cdot \text{B1E1} = 288.085$$

$$\text{B1E2} := \int_0^{5138.4208} 104.57235 \cdot \exp(-0.0001922t) \, dt$$

$$\text{B1E2} = 3.414 \times 10^5 \text{ C}$$

$$\text{Con} \cdot \text{B1E2} = 355.716$$

BEAM 2

$$\text{B2E1} := \int_0^{5138.4208} 109.09371 \cdot \exp(-0.00040747t) \, dt$$

$$\text{B2E1} = 2.347 \times 10^5$$

$$\text{Con} \cdot \text{B2E1} = 244.566 \text{ g}$$

$$\text{B2E2} := \int_0^{5138.4208} 106.64603 \exp(-0.00018825t) \, dt$$

$$\text{B2E2} = 3.512 \times 10^5$$

$$\text{Con} \cdot \text{B2E2} = 365.874$$

BEAM 3

$$B3E1 := \int_0^{5138.4208} 115.148898 \exp(-0.00017982t) dt$$

$$B3E1 = 3.862 \times 10^5 \text{ C}$$

$$\text{Con} \cdot B3E1 = 402.339 \text{ g}$$

$$B3E2 := \int_0^{5138.4208} 107.9592 \exp(-0.00016802t) dt$$

$$B3E2 = 3.716 \times 10^5$$

$$\text{Con} \cdot B3E2 = 387.099$$

BEAM 4

$$B4E1 := \int_0^{5138.4208} 114.450087 \exp(-0.00019347t) dt$$

$$B4E1 = 3.727 \times 10^5 \text{ C}$$

$$\text{Con} \cdot B4E1 = 388.254$$

$$B4E2 := \int_0^{5138.4208} 123.723 \exp(-0.00022095t) dt$$

$$B4E2 = 3.8 \times 10^5 \text{ C}$$

$$\text{Con} \cdot B4E2 = 395.939$$

BEAM 5

$$B5E1 := \int_0^{5138.4208} 114.3122 \exp(-0.00018556t) \, dt$$

$$B5E1 = 3.786 \times 10^5 \, \text{C}$$

$$\text{Con} \cdot B5E1 = 394.464 \, \text{g}$$

$$B5E2 := \int_0^{5138.4208} 118.23531 \exp(-0.000178155t) \, dt$$

$$B5E2 = 3.98 \times 10^5 \, \text{C}$$

$$\text{Con} \cdot B5E2 = 414.626 \, \text{g}$$

APPENDIX F

Half-Cell Data & Contours

Half-Cell Potential Measurements

Date: 4/11/02 and 4/12/02

Temperature:

Beam 1

Position/Point	1	2	3	4	5	6	7	8	9	10	11	12	Center
x-position	1	7	13	19	1	7	13	19	1	7	13	19	48
y-position	19.5	19.5	19.5	19.5	9	9	9	9	3	3	3	3	19.5
SW													
NW													
SE	0.226	0.230	0.231	0.220	0.236	0.237	0.231	0.241	0.230	0.243	0.245	0.240	0.223
NE	0.233	0.241	0.234	0.229	0.310	0.247	0.233	0.227	0.250	0.243	0.252	0.233	0.225

Beam 2

Position/Point	1	2	3	4	5	6	7	8	9	10	11	12	Center
SW	0.229	0.229	0.231	0.220	0.258	0.243	0.238	0.234	0.250	0.241	0.235	0.231	0.165
NW	0.215	0.218	0.219	0.197	0.273	0.247	0.235	0.227	0.233	0.237	0.225	0.212	0.190
SE	0.207	0.221	0.206	0.213	0.215	0.219	0.212	0.214	0.179	0.204	0.226	0.226	0.165
NE	0.208	0.212	0.213	0.205	0.276	0.233	0.212	0.208	0.221	0.223	0.218	0.205	0.190

Beam 3

Position/Point	1	2	3	4	5	6	7	8	9	10	11	12	Center
SW													
NW													
SE	0.205	0.235	0.251	0.239	0.283	0.244	0.244	0.255	0.225	0.222	0.226	0.243	0.220
NE	0.222	0.247	0.241	0.238	0.274	0.252	0.244	0.245	0.213	0.215	0.229	0.210	0.229

Beam 4

Position/Point	1	2	3	4	5	6	7	8	9	10	11	12	Center
SW	0.242	0.247	0.252	0.249	0.270	0.259	0.242	0.252	0.222	0.245	0.230	0.244	0.250
NW	0.184	0.206	0.211	0.197	0.214	0.211	0.218	0.230	0.184	0.199	0.208	0.211	0.250
SE	0.236	0.251	0.252	0.249	0.260	0.257	0.253	0.253	0.232	0.240	0.250	0.238	0.250
NE	0.201	0.218	0.210	0.217	0.224	0.221	0.223	0.220	0.181	0.190	0.193	0.221	0.250

Half-Cell Potential Measurements

Date: 5/29/02

Temperature:

Beam 1

Position/Point	1	2	3	4	5	6	7	8	9	10	11	12	Center
SW													
NW													
SE	0.094	0.089	0.062	0.036	0.176	0.132	0.118	0.114	0.202	0.146	0.141	0.126	0.020
NE	0.111	0.089	0.066	0.047	0.239	0.144	0.131	0.117	0.280	0.176	0.165	0.146	0.013

Beam 2

Position/Point	1	2	3	4	5	6	7	8	9	10	11	12	Center
SW	0.209	0.182	0.136	0.103	0.321	0.248	0.241	0.213	0.349	0.260	0.260	0.248	0.017
NW	0.224	0.195	0.156	0.111	0.293	0.264	0.256	0.219	0.335	0.300	0.303	0.244	0.005
SE	0.159	0.108	0.082	0.051	0.229	0.163	0.138	0.119	0.273	0.174	0.160	0.154	0.017
NE	0.203	0.161	0.100	0.065	0.330	0.211	0.174	0.156	0.303	0.214	0.198	0.194	0.005

Beam 3

Position/Point	1	2	3	4	5	6	7	8	9	10	11	12	Center
SW													
NW													
SE	0.348	0.261	0.273	0.253	0.537	0.413	0.390	0.389	0.542	0.441	0.447	0.409	0.155
NE	0.283	0.292	0.246	0.241	0.476	0.401	0.378	0.369	0.472	0.424	0.426	0.415	0.148

Beam 4

Position/Point	1	2	3	4	5	6	7	8	9	10	11	12	Center
SW	0.244	0.209	0.174	0.152	0.411	0.274	0.254	0.236	0.346	0.292	0.302	0.283	0.106
NW	0.280	0.210	0.183	0.178	0.358	0.300	0.256	0.244	0.336	0.288	0.281	0.278	0.110
SE	0.267	0.233	0.208	0.170	0.379	0.301	0.280	0.275	0.371	0.330	0.338	0.323	0.106
NE	0.205	0.197	0.166	0.155	0.329	0.256	0.222	0.201	0.367	0.273	0.230	0.219	0.100

Half-Cell Potential Measurements

Date: 7/22/02

Temperature: 85 outside

Beam 1

Position/Point	1	2	3	4	5	6	7	8	9	10	11	12	Center
SW													
NW													
SE	242.2	209.1	170.1	143.2	411.0	296.4	244.0	224.3	432.0	312.4	284.8	253.1	65.3
NE	281.0	216.3	173.0	122.0	467.0	336.4	256.4	231.0	502.0	346.0	299.1	260.1	56.7

Beam 2

Position/Point	1	2	3	4	5	6	7	8	9	10	11	12	Center
SW	294.4	244.8	183.1	150.3	427.0	371.5	299.6	255.1	518.0	396.3	297.5	266.7	22.4
NW	293.0	238.5	189.1	153.0	409.0	353.8	289.0	255.9	572.0	392.0	361.1	301.3	45.6
SE	278.7	204.9	155.4	115.0	380.0	308.0	222.1	185.0	443.0	285.9	223.6	219.9	22.4
NE	322.8	236.1	154.6	110.2	415.0	290.4	343.7	198.0	466.0	308.8	265.4	243.8	45.6

Beam 3

Position/Point	1	2	3	4	5	6	7	8	9	10	11	12	Center
SW													
NW													
SE	360.0	239.5	232.6	219.2	520.0	408.0	331.1	327.0	509.0	395.6	372.1	339.1	129.3
NE	306.3	280.2	215.4	213.2	485.0	407.5	353.3	358.1	505.0	448.0	406.0	415.0	101.0

Beam 4

Position/Point	1	2	3	4	5	6	7	8	9	10	11	12	Center
SW	330.0	264.0	214.2	185.7	486.0	347.1	305.8	284.3	527.0	365.5	369.1	338.1	104.5
NW	331.0	215.7	196.6	191.6	470.0	346.0	289.2	274.5	400.4	355.0	322.1	301.4	121.0
SE	310.9	238.4	192.6	183.1	396.0	281.5	264.0	286.9	440.0	292.4	303.0	312.3	104.5
NE	309.4	226.5	190.4	174.8	444.0	314.9	266.8	269.7	476.0	355.0	283.2	274.0	121.0

Half-Cell Potential Measurements

Date: 9/17/02

Temperature: 70-80's (outside)

Beam 1

Position/Point	1	2	3	4	5	6	7	8	9	10	11	12	Center
SW													
NW													
SE	138.0	79.6	46.2	20.0	237.5	157.0	111.5	99.9	296.0	184.6	121.1	105.8	-35.0
NE	146.1	75.1	43.0	19.0	235.1	158.0	143.9	87.4	255.0	153.3	136.5	112.9	-50.0

Beam 2

Position/Point	1	2	3	4	5	6	7	8	9	10	11	12	Center
SW	275.5	192.0	113.6	66.0	359.6	301.8	216.6	159.5	468.0	344.1	220.3	168.4	-44.0
NW	221.0	180.0	99.5	60.5	324.4	289.1	203.7	165.6	374.5	310.4	240.5	193.2	-44.3
SE	204.1	132.5	70.0	24.5	248.0	205.7	112.9	94.2	285.4	207.2	143.5	127.0	-44.0
NE	219.1	160.9	66.7	42.2	263.4	178.1	130.4	109.3	325.5	177.6	132.1	112.6	-44.3

Beam 3

Position/Point	1	2	3	4	5	6	7	8	9	10	11	12	Center
SW													
NW													
SE	293.7	204.0	166.0	138.0	363.7	295.4	255.0	254.9	350.3	305.4	274.7	232.9	26.8
NE	258.2	245.3	143.1	148.4	419.0	362.3	309.4	306.5	421.0	450.0	376.3	393.0	45.0

Beam 4

Position/Point	1	2	3	4	5	6	7	8	9	10	11	12	Center
SW	288.1	255.0	153.0	107.5	309.4	276.6	223.3	187.9	301.8	291.5	268.0	223.0	33.3
NW	282.3	181.5	153.1	113.3	330.7	253.0	217.4	183.6	312.7	296.6	232.5	207.1	31.5
SE	163.4	130.0	77.2	38.0	208.4	157.5	129.7	141.1	145.1	160.6	228.8	177.5	33.3
NE	141.3	122.3	57.9	42.3	189.3	140.5	124.0	140.2	224.7	186.1	129.0	126.6	31.5

Half-Cell Potential Measurements

Date: 10/29/03

Temperature: 40 (outside)

Beam 1

Position/Point	1	2	3	4	5	6	7	8	9	10	11	12	Center
SW													
NW													
SE	121.2	74.8	11.3	-32.0	186.6	103.4	79.9	71.3	238.6	172.0	76.2	61.1	-93.0
NE	135.7	84.5	3.9	-43.8	165.6	150.4	109.1	50.5	204.1	152.6	92.3	55.7	-101.1

Beam 2

Position/Point	1	2	3	4	5	6	7	8	9	10	11	12	Center
SW	187.0	173.0	86.2	41.9	258.7	230.6	163.1	119.0	376.6	258.4	156.7	126.5	-45.3
NW	197.5	162.2	79.6	43.7	249.5	250.4	158.5	111.6	272.5	232.7	159.9	121.4	-36.7
SE	312.8	262.8	120.9	48.7	343.0	252.3	162.8	145.1	410.0	308.6	205.9	191.6	-45.3
NE	281.6	240.6	115.0	56.0	330.7	227.5	172.8	143.5	366.4	292.8	172.7	148.3	-36.7

Beam 3

Position/Point	1	2	3	4	5	6	7	8	9	10	11	12	Center
SW													
NW													
SE	265.0	187.7	116.8	93.0	390.2	325.9	253.0	234.6	268.5	333.9	336.1	217.0	-5.0
NE	283.8	247.7	163.2	130.7	429.0	397.5	334.5	316.0	462.0	471.0	437.0	389.2	18.0

Beam 4

Position/Point	1	2	3	4	5	6	7	8	9	10	11	12	Center
SW	194.5	184.6	108.3	68.1	246.0	233.4	180.6	142.0	244.5	228.5	194.1	190.6	-35.5
NW	204.5	141.2	94.3	65.4	277.4	234.3	162.4	133.4	270.4	267.4	193.5	143.5	-23.0
SE	70.9	61.7	21.0	-1.0	153.7	114.8	74.5	80.7	154.5	126.7	96.4	89.5	-35.5
NE	90.9	77.2	20.4	5.0	182.8	98.2	82.4	103.9	186.3	164.9	94.3	81.7	-23.0

Half-Cell Potential Measurements

Date: 3/25/03
 Temperature: 55 outside

Beam 2

Position/Point	1	2	3	4	5	6	7	8	9	10	11	12	Center
SW	265.7	245.5	208.8	27.1	302.9	266.7	235.1	209.7	308.5	275.0	254.3	216.8	-10.6
NW	277.0	292.0	219.2	101.5	358.0	272.6	255.2	203.7	337.7	270.4	249.5	219.3	-12.3
SE	495.0	398.2	287.9	246.1	533.0	505.0	475.0	397.0	541.0	525.0	515.0	449.0	-10.6
NE	411.0	335.2	289.8	232.3	391.7	462.0	425.0	428.0	485.0	488.0	483.0	459.0	-12.3

Half-Cell Potential Measurements

Date: 5/5/03

Temperature: 50 outside

Beam 2

Position/Point	1	2	3	4	5	6	7	8	9	10	11	12	Center
SW	336.1	295.4	250.5	212.0	414.0	351.7	298.1	264.2	434.0	361.5	309.6	271.5	-34.4
NW	336.6	319.6	247.6	117.2	405.7	348.7	298.7	268.0	451.0	369.5	310.7	276.5	-32.2
SE	519.0	428.0	321.7	255.3	540.0	510.0	479.0	393.1	556.0	524.0	511.0	486.0	-34.4
NE	453.0	395.9	312.7	278.7	462.0	457.0	445.0	417.0	510.0	511.0	493.0	445.0	-32.2

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Position/Point	5
Beam 1 (coating)	207.0
Beam 3 (sealer)	426.0
Beam 5 (polymer)	-240.0
Beam 5 (FRP)	-320.0

Measurement with Plastic under electrode = 0

Half-Cell Potential Measurements

Date: 12/21/03

Beam 2

Position/Point	1	2	3	4	5	6	7	8	9	10	11	12	Center
SW	430.0	350.0	319.0	259.0	433.0	334.0	297.0	240.0	406.0	371.0	315.0	268.0	-107.0
NW	525.0	397.0	345.0	279.0	497.0	415.0	346.0	262.0	418.0	388.0	348.0	326.0	-133.0
SE	532.0	433.0	346.0	293.0	489.0	481.0	432.0	400.0	476.0	501.0	516.0	441.0	-107.0
NE	503.0	431.0	357.0	294.0	449.0	453.0	413.0	353.0	432.0	464.0	450.0	401.0	-133.0

Contours for 4-11-2002 Measurements

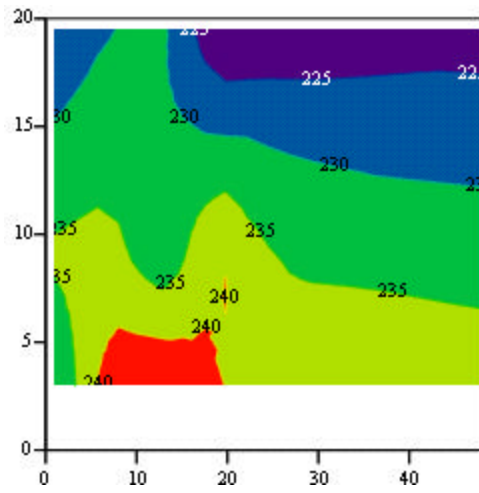


Figure F1. Beam 1: Southeast End

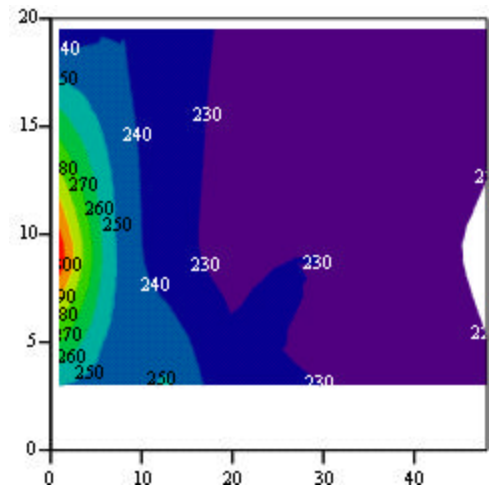


Figure F2. Beam 1: Northeast End

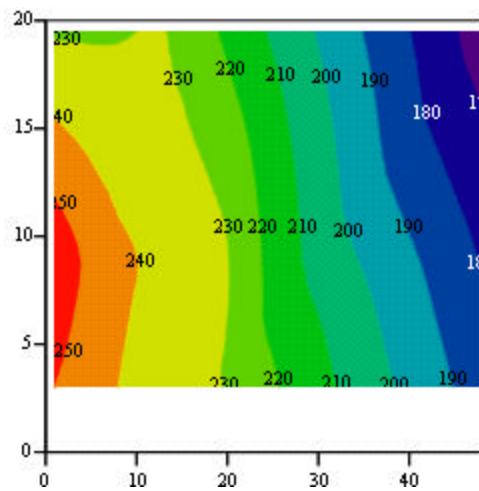


Figure F3. Beam 2: Southwest End

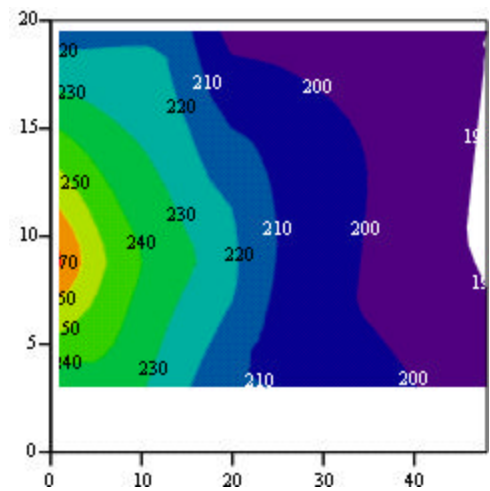


Figure F4. Beam 2: Northwest End

Contours for 4-11-2002 Measurements

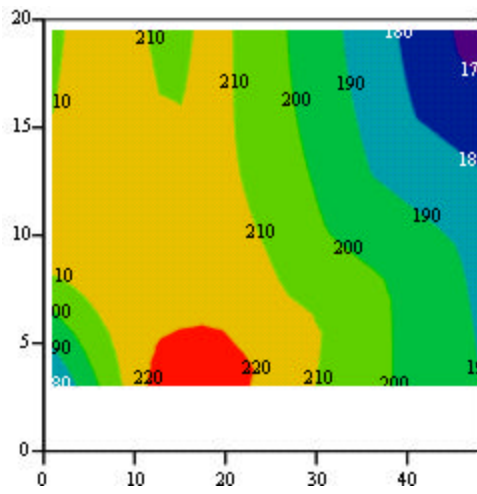


Figure F5. Beam 2: Southeast End

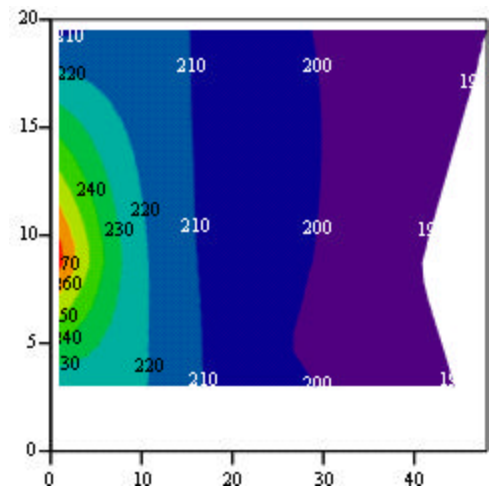


Figure F6. Beam 2: Northeast End

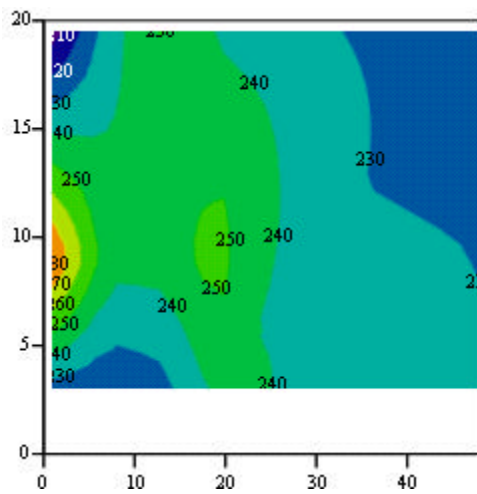


Figure F7. Beam 3: Southeast End

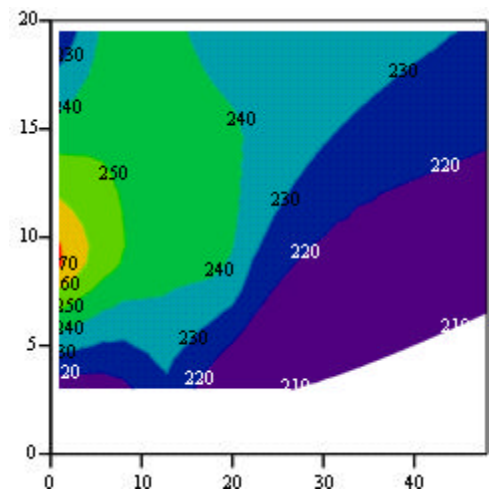


Figure F8. Beam 3: Northeast End

Contours for 4-11-2002 Measurements

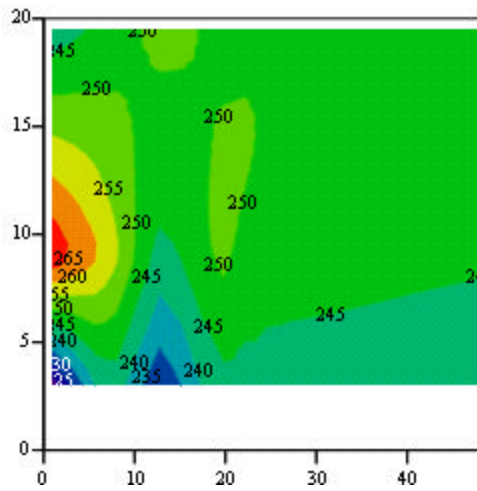


Figure F9. Beam 4: Southwest End

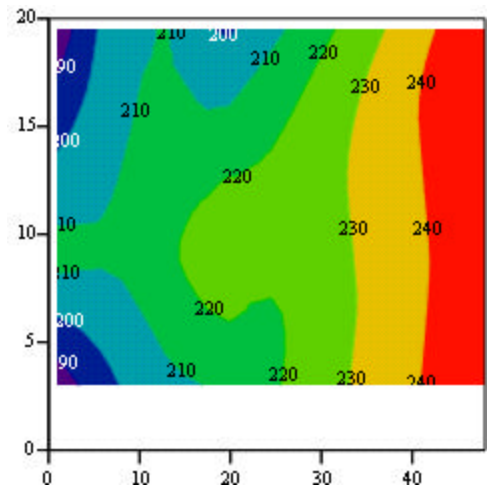


Figure F10. Beam 4: Northwest End

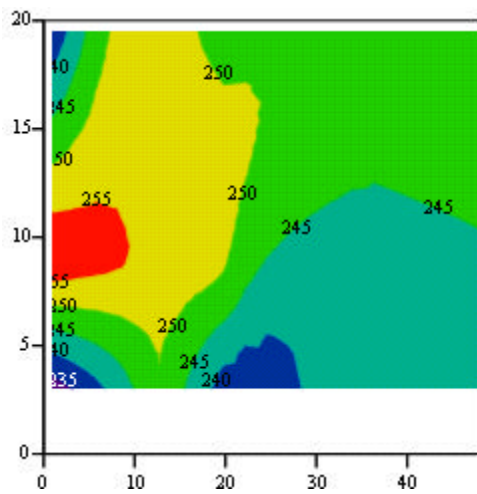


Figure F11. Beam 4: Southeast End

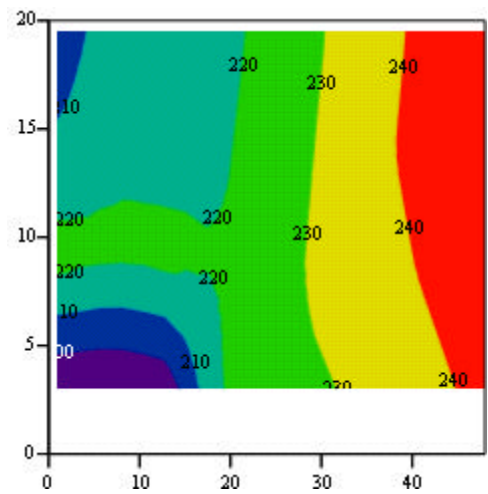


Figure F12. Beam 4: Northeast End

Contours for 5-29-2002 Measurements

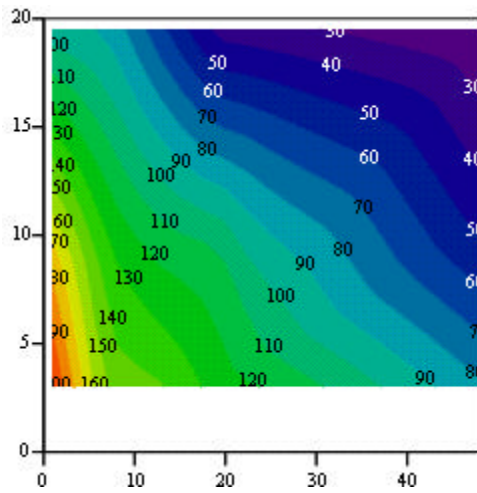


Figure F13. Beam 1: Southeast End

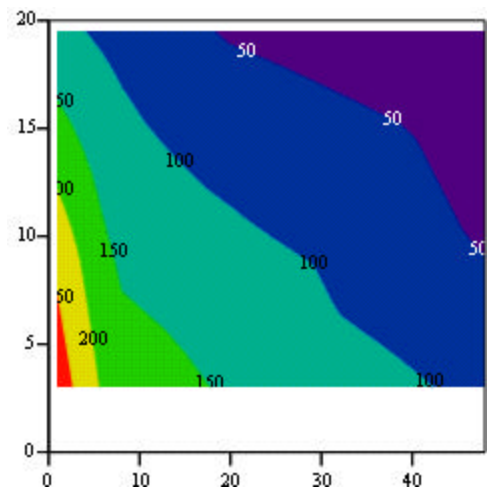


Figure F14. Beam 1: Northeast End

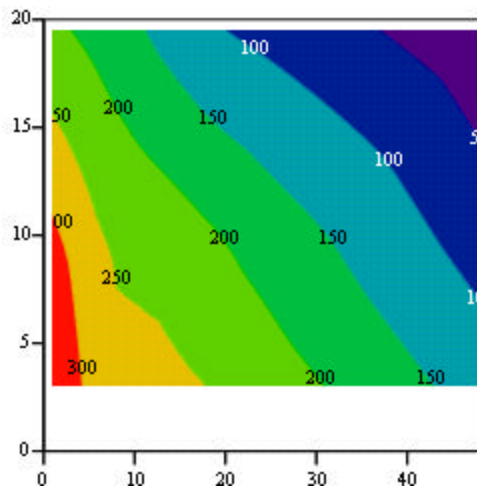


Figure F15. Beam 2: Southwest End

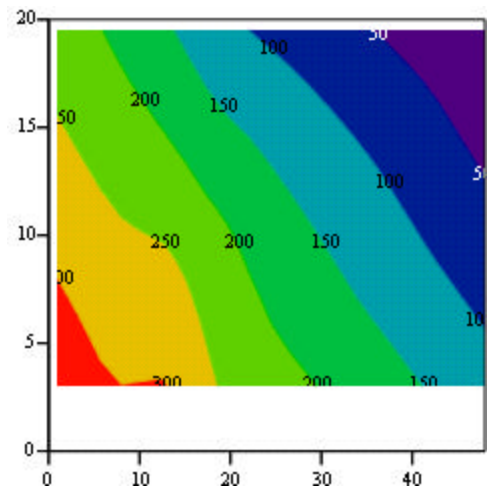


Figure F16. Beam 2: Northwest End

Contours for 5-29-2002 Measurements

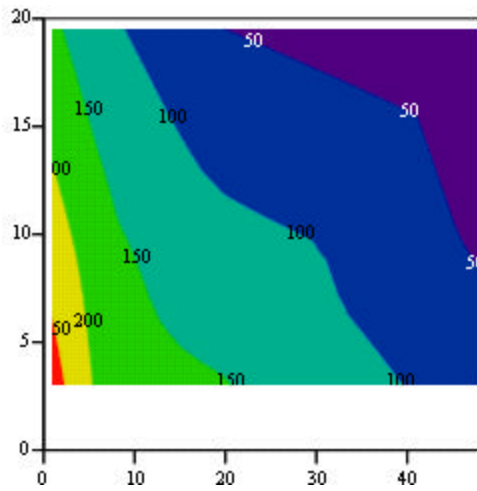


Figure F17. Beam 2: Southeast End

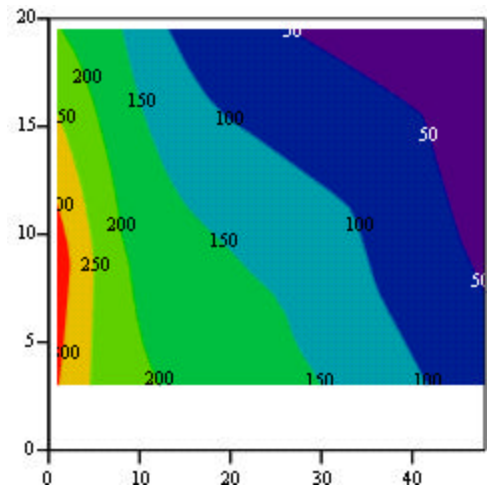


Figure F18. Beam 2: Northeast End

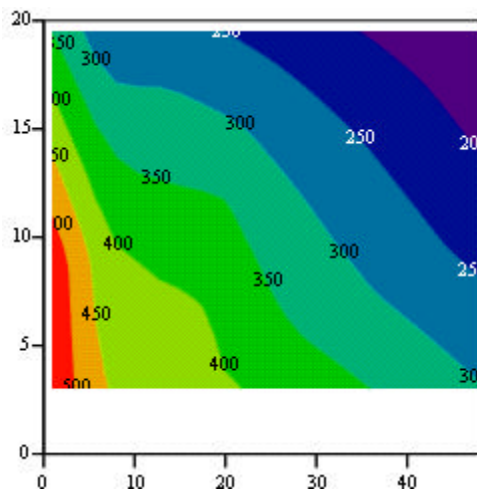


Figure F19. Beam 3: Southeast End

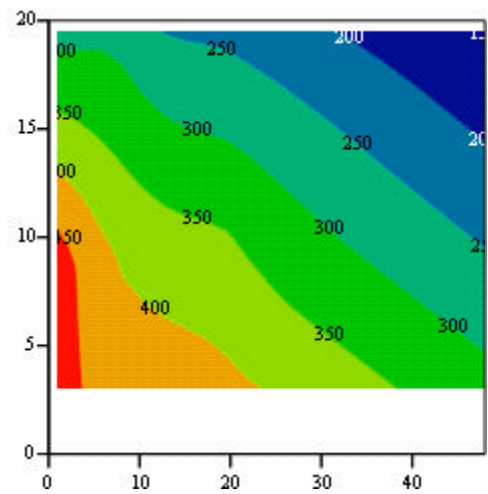


Figure F20. Beam 3: Northeast End

Contours for 5-29-2002 Measurements

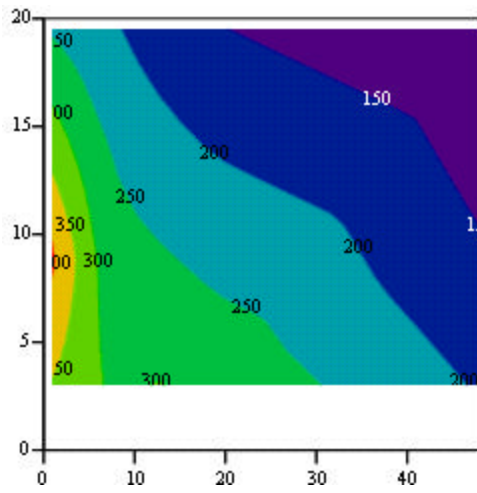


Figure F21. Beam 4: Southwest End

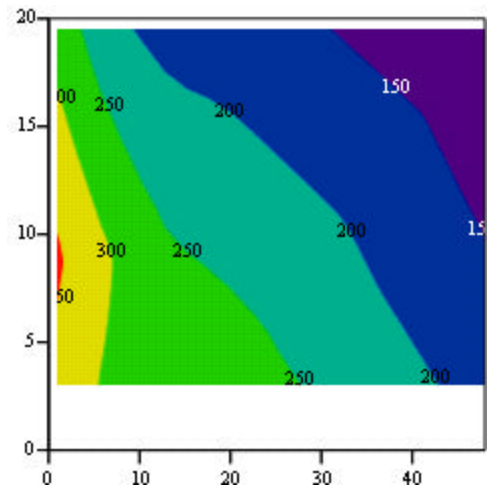


Figure F22. Beam 4: Northwest End

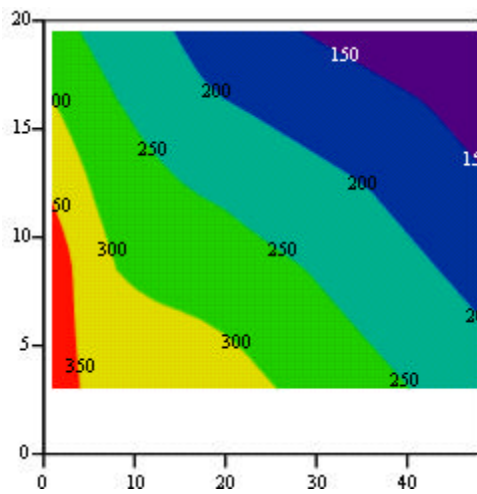


Figure F23. Beam 4: Southeast End

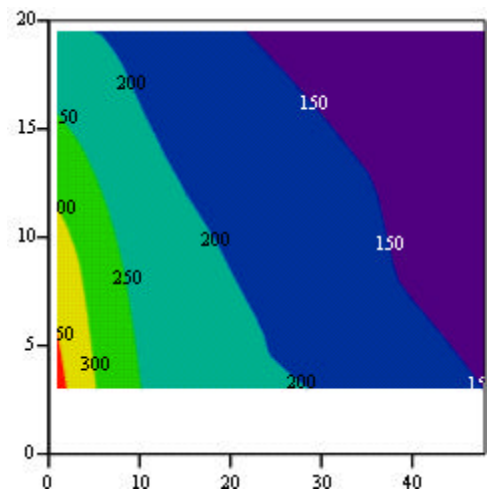


Figure F24. Beam 4: Northeast End

Contours for 7-22-2002 Measurements

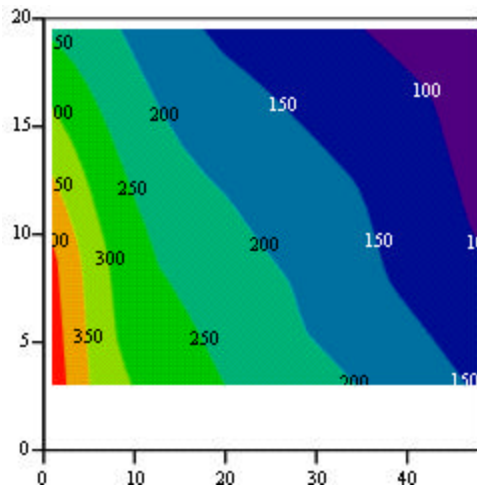


Figure F25. Beam 1: Southeast End

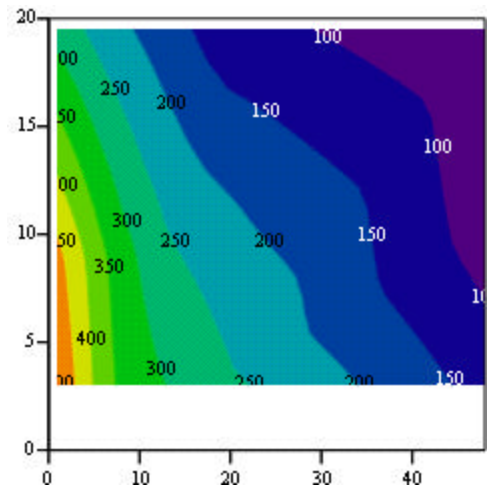


Figure F26. Beam 1: Northeast End

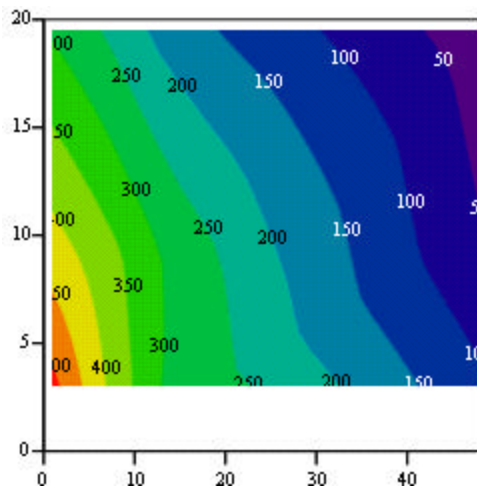


Figure F27. Beam 2: Southwest End

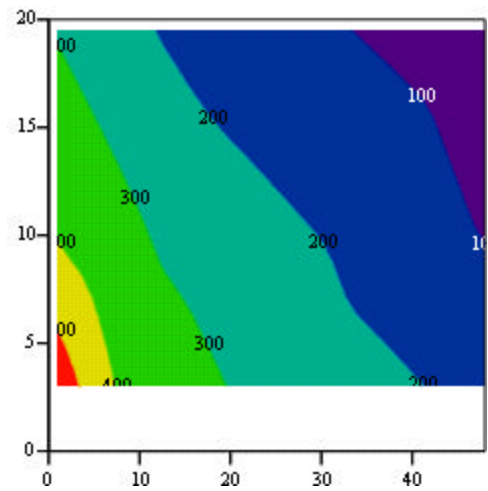


Figure F28. Beam 2: Northwest End

Contours for 7-22-2002 Measurements

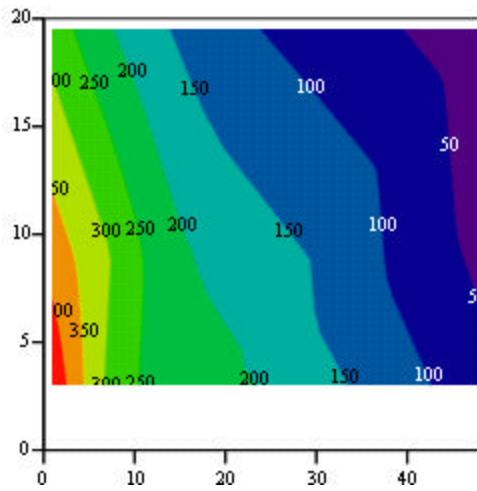


Figure F29. Beam 2: Southeast End

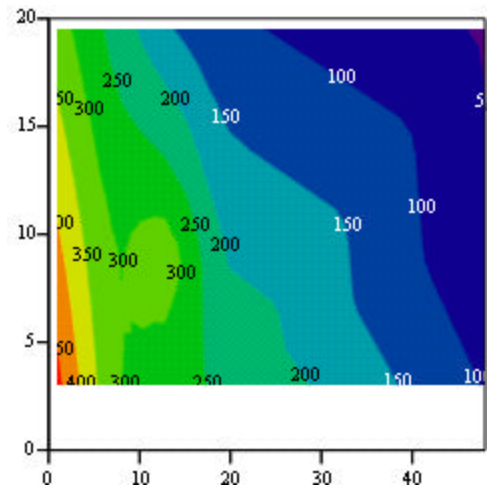


Figure F30. Beam 2: Northeast End

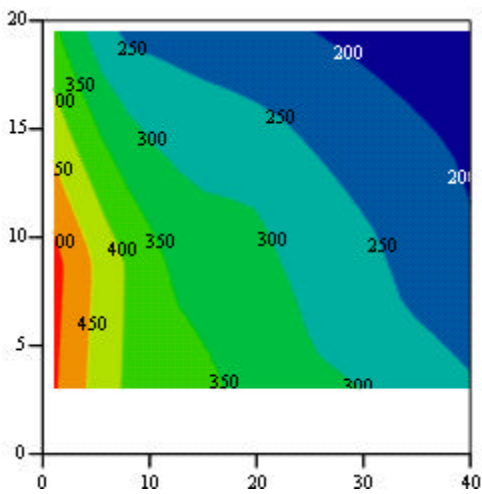


Figure F31. Beam 3: Southeast End

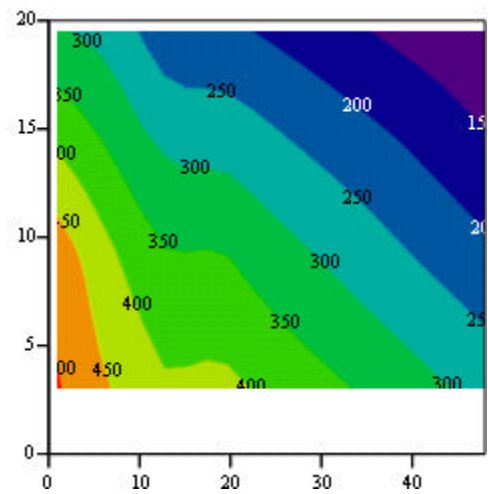


Figure F32. Beam 3: Northeast End

Contours for 7-22-2002 Measurements

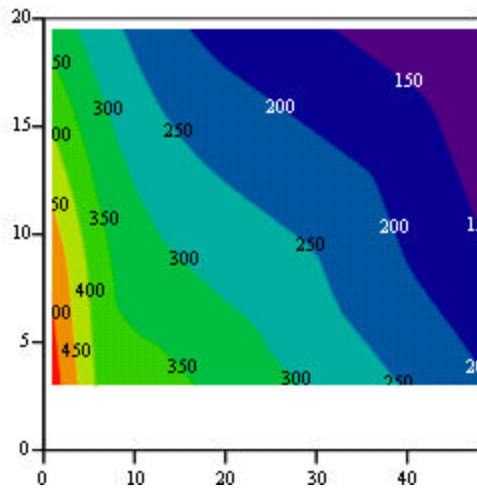


Figure F33. Beam 4: Southwest End

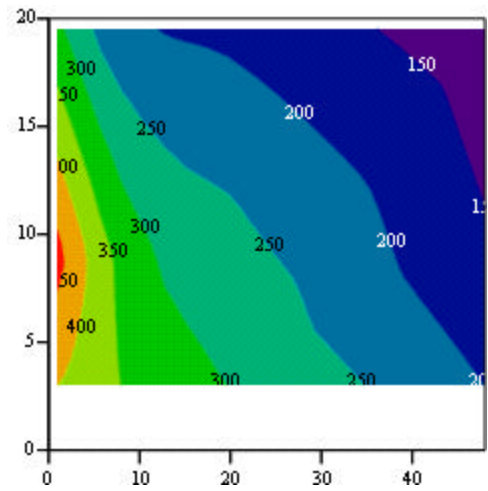


Figure F34. Beam 4: Northwest End

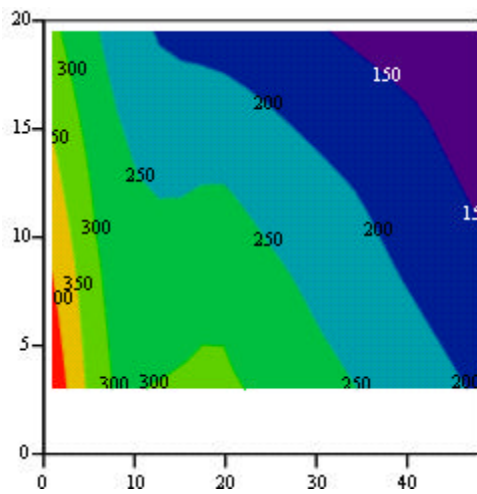


Figure F35. Beam 4: Southeast End

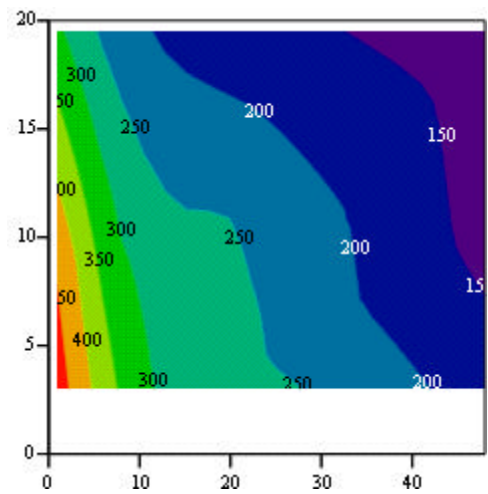


Figure F36. Beam 4: Northeast End

Contours for 9-17-2002 Measurements

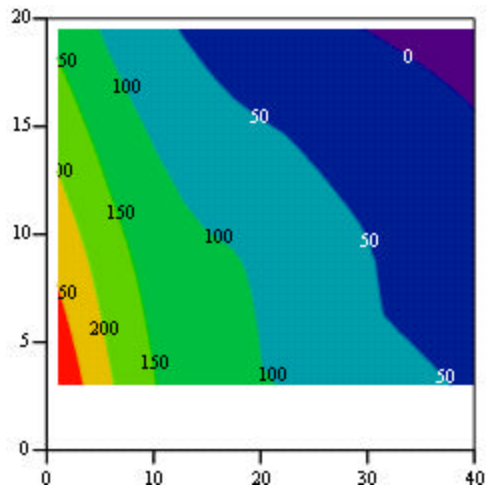


Figure F37. Beam 1: Southeast End

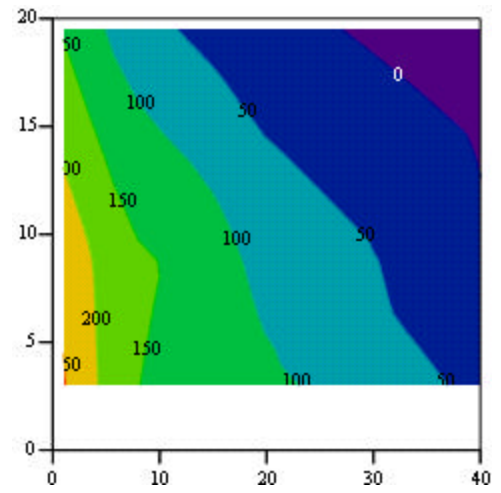


Figure F38. Beam 1: Northeast End

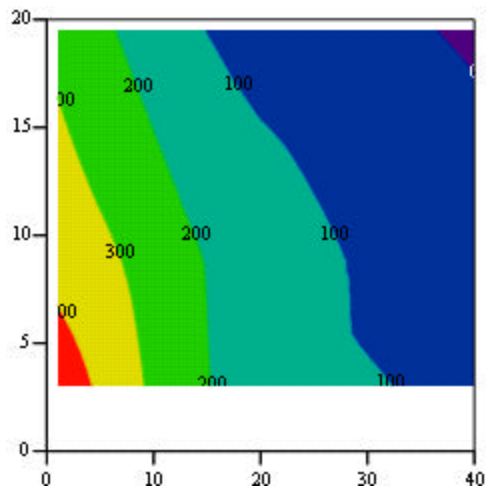


Figure F39. Beam 2: Southwest End

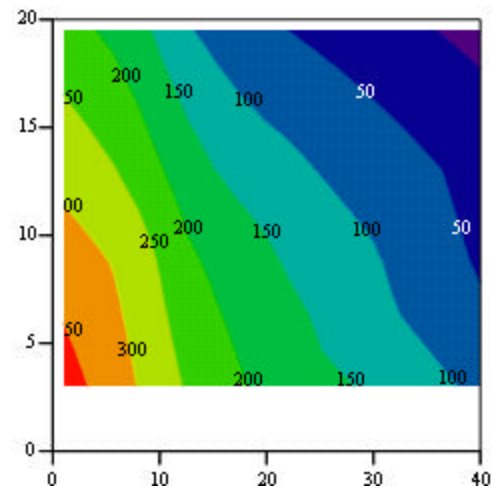


Figure F40. Beam 2: Northwest End

Contours for 9-17-2002 Measurements

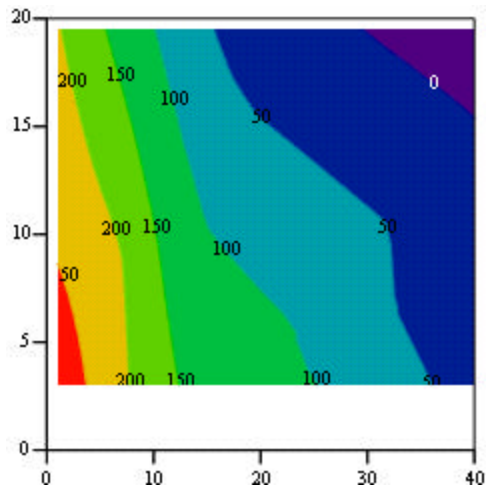


Figure F41. Beam 2: Southeast End

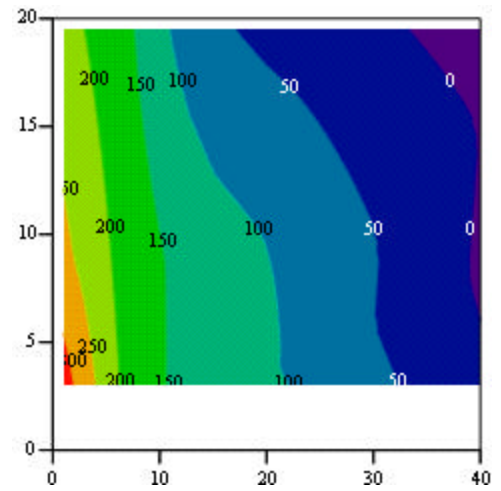


Figure F42. Beam 2: Northeast End

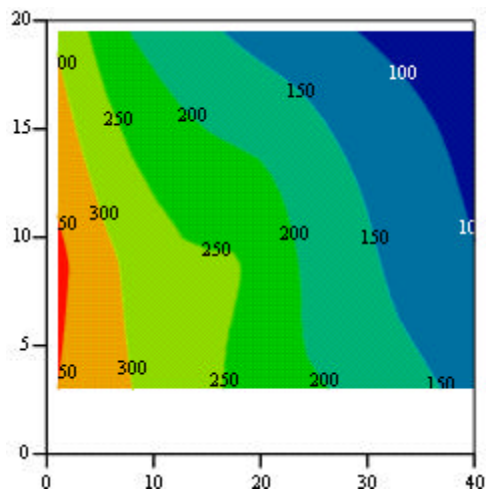


Figure F43. Beam 3: Southeast End

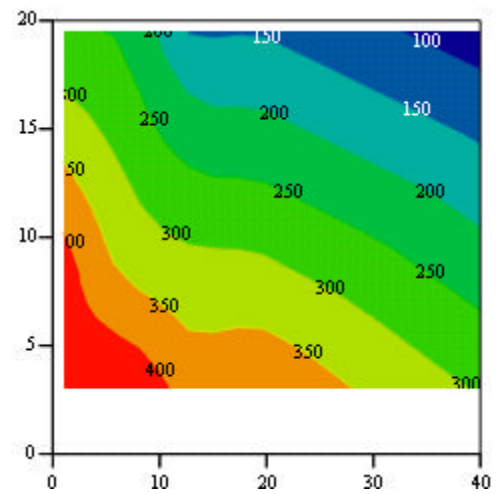


Figure F44. Beam 3: Northeast End

Contours for 9-17-2002 Measurements

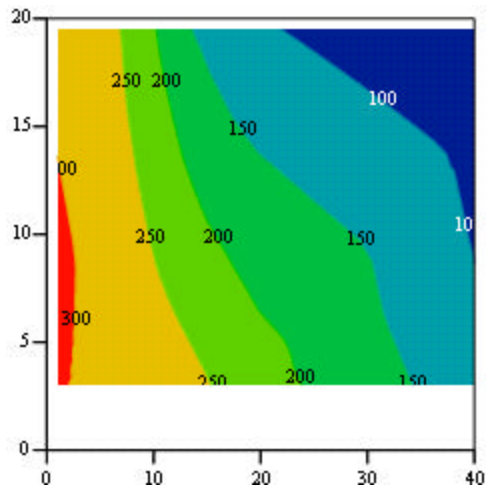


Figure F45. Beam 4: Southwest End

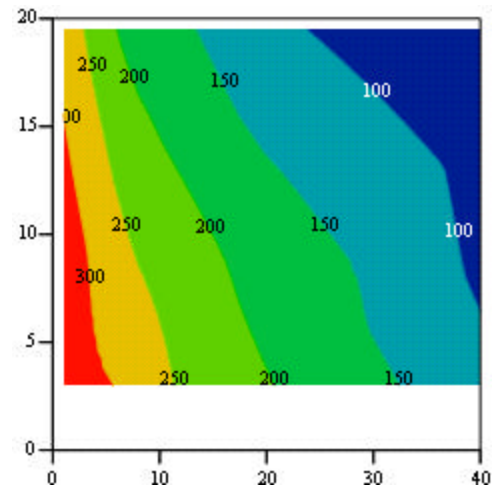


Figure F46. Beam 4: Northwest End

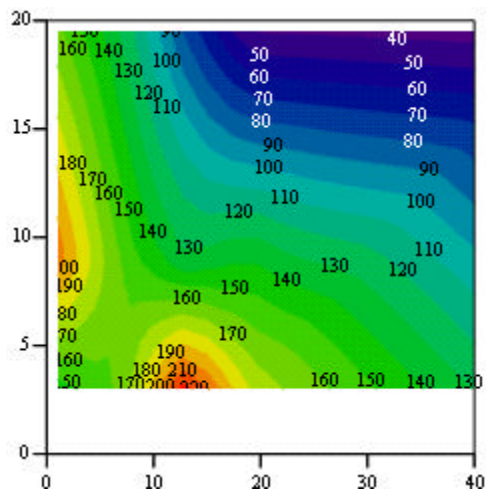


Figure F47. Beam 4: Southeast End

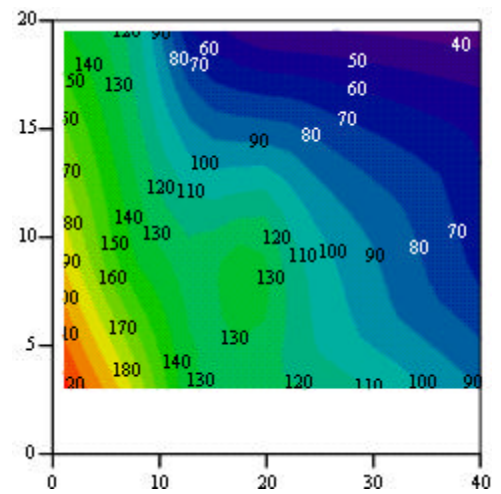


Figure F48. Beam 4: Northeast End

Contours for 10-29-2002 Measurements

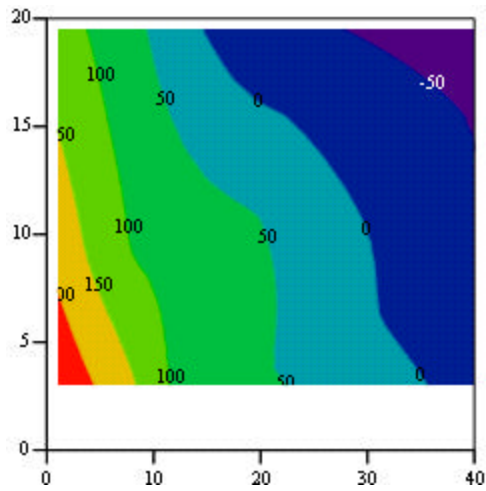


Figure F49. Beam 1: Southeast End

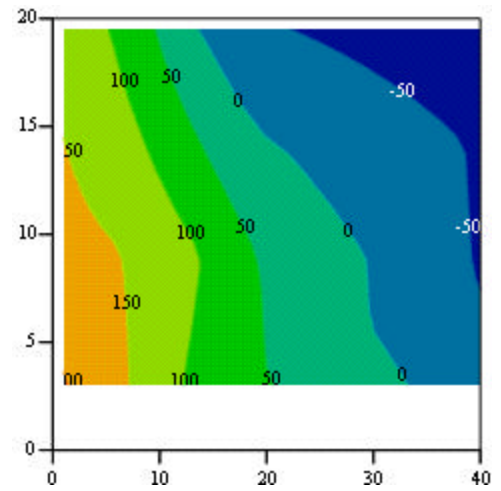


Figure F50. Beam 1: Northeast End

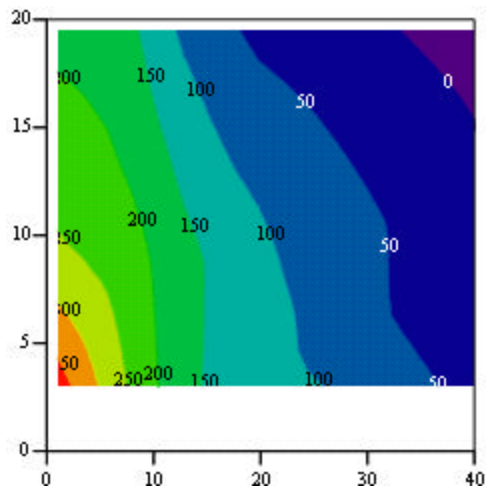


Figure F51. Beam 2: Southwest End

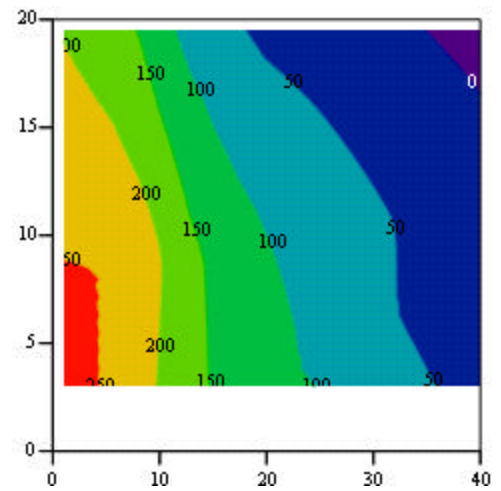


Figure F52. Beam 2: Northwest End

Contours for 10-29-2002 Measurements

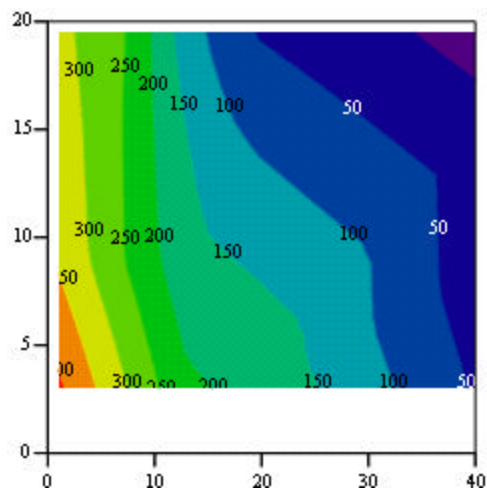


Figure F53. Beam 2: Southeast End

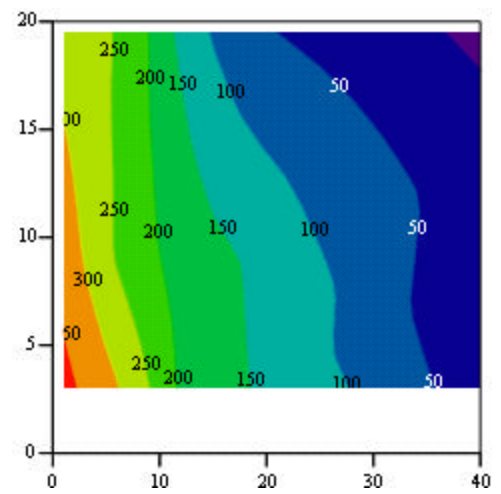


Figure F54. Beam 2: Northeast End

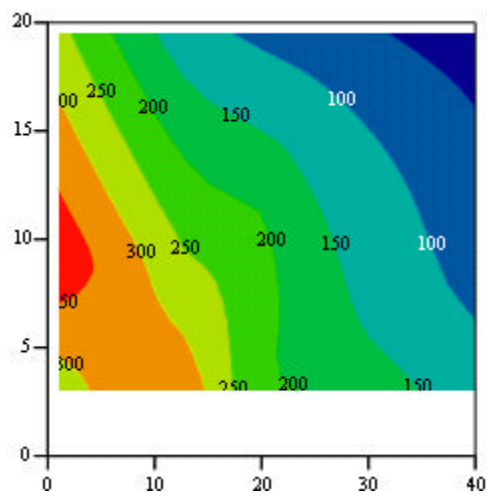


Figure F55. Beam 3: Southeast End

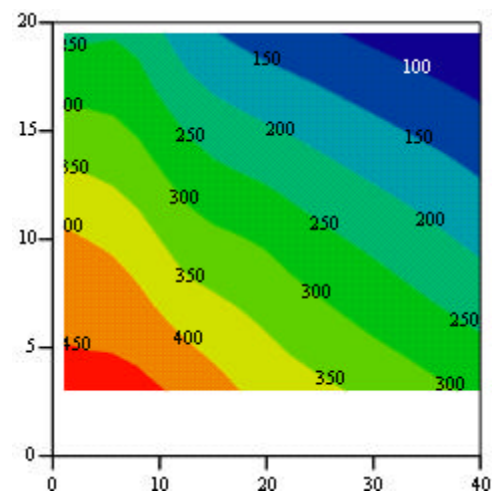


Figure F56. Beam 3: Northeast End

Contours for 10-29-2002 Measurements

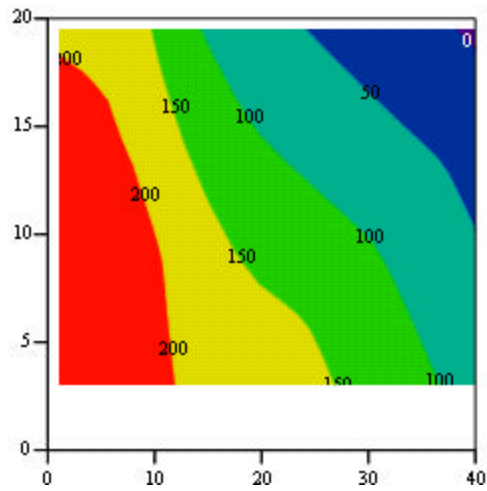


Figure F57. Beam 4: Southwest End

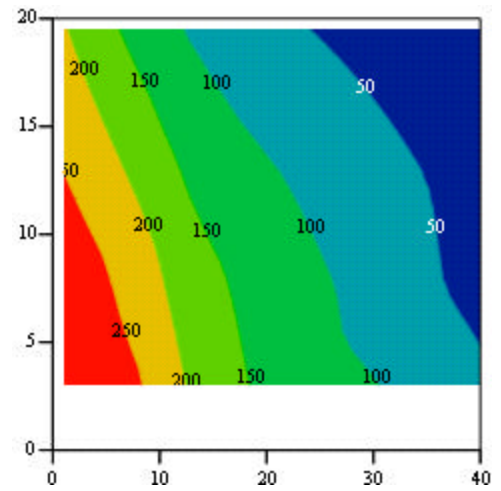


Figure F58. Beam 4: Northwest End

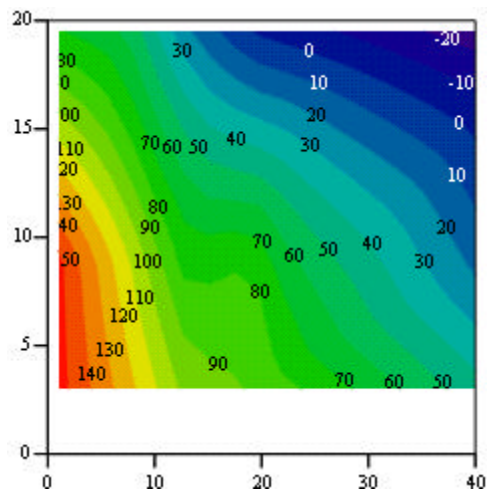


Figure F59. Beam 4: Southeast End

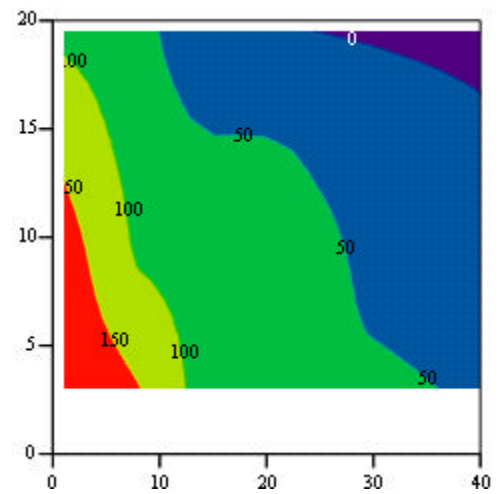


Figure F60. Beam 4: Northeast End

Contours for 3-25-2003 Measurements

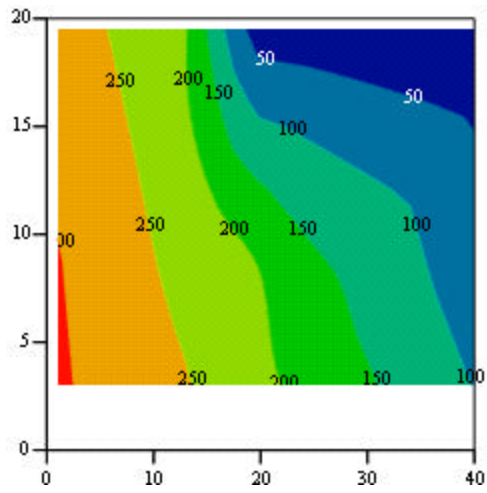


Figure F61. Beam 2: Southwest End

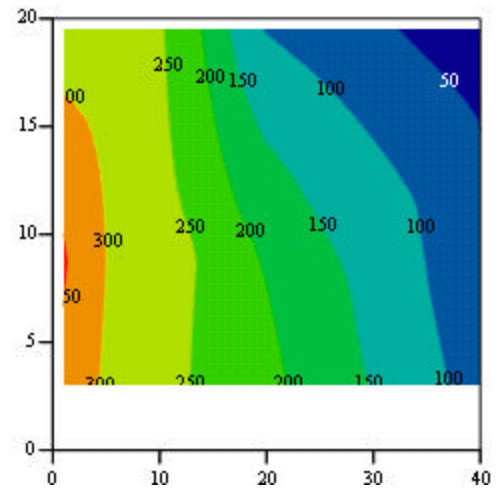


Figure F62. Beam 2: Northwest End

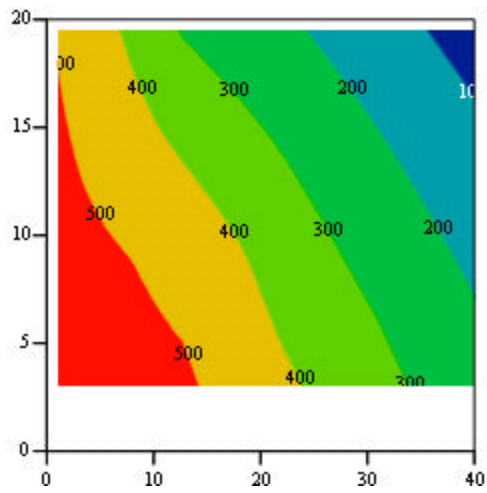


Figure F63. Beam 2: Southeast End

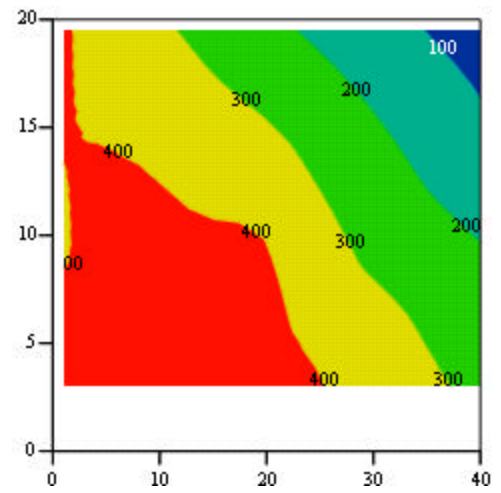


Figure F64. Beam 2: Northeast End

Contours for 5-5-2003 Measurements

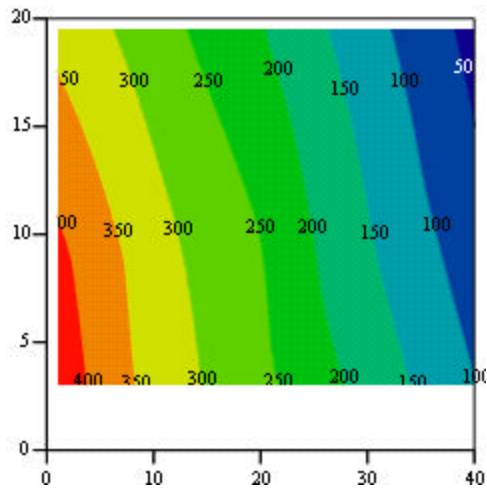


Figure F65. Beam 2: Southwest End

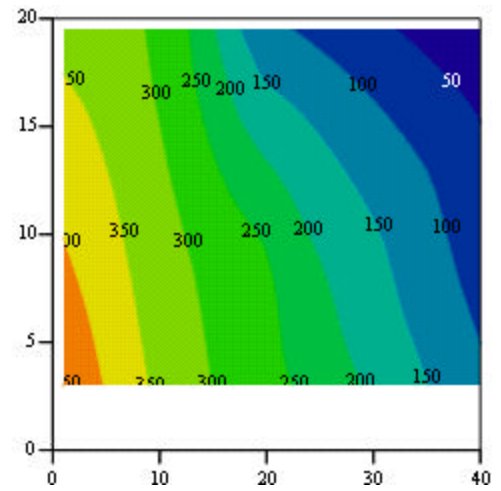


Figure F66. Beam 2: Northwest End

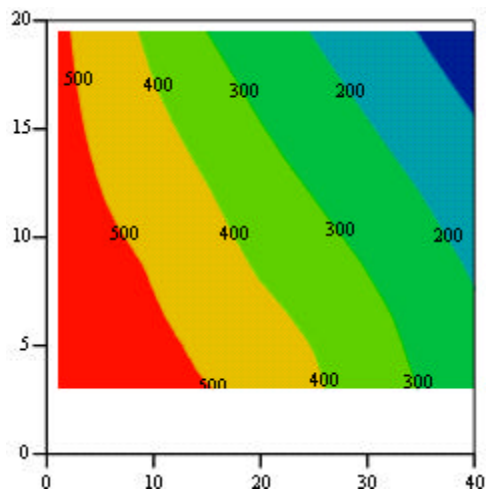


Figure F67. Beam 2: Southeast End

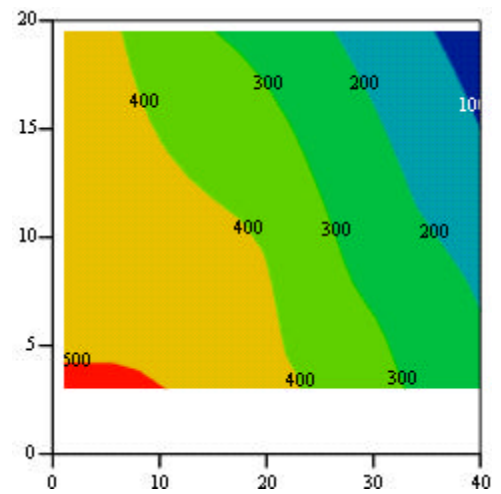


Figure F68. Beam 2: Northeast End

Contours for 12-21-2003 Measurements

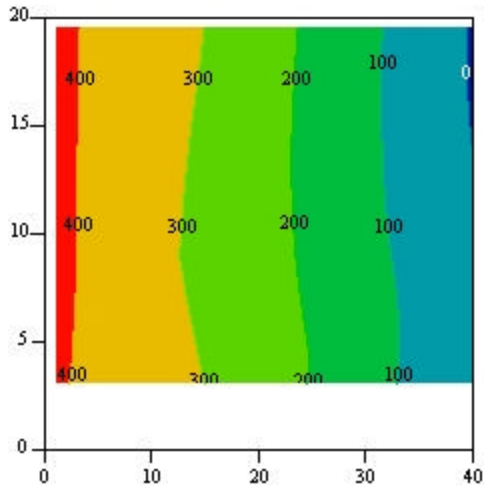


Figure F69. Beam 2: Southwest End

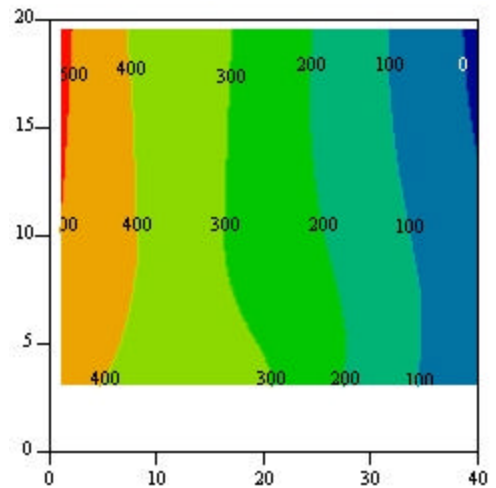


Figure F70. Beam 2: Northwest End

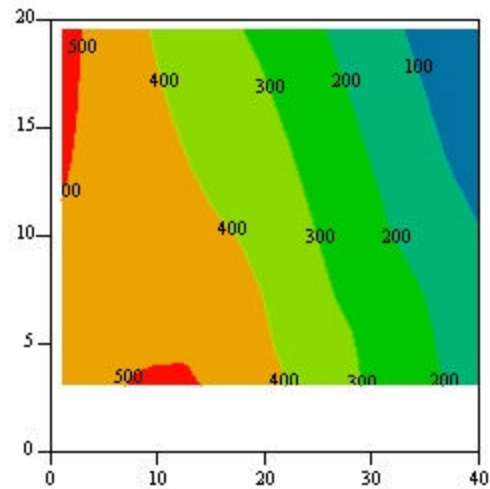


Figure F71. Beam 2: Southeast End

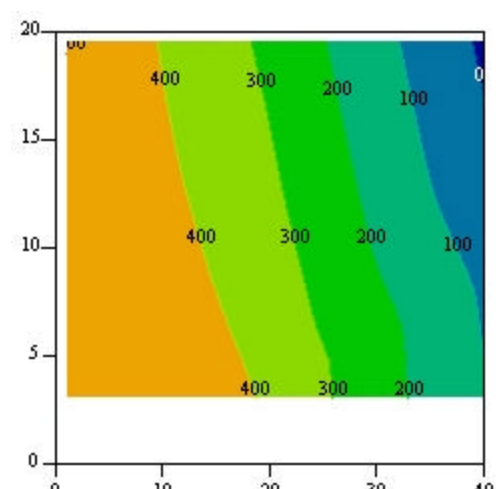


Figure F72. Beam 2: Northeast End